The behaviour of full-scale steel-framed buildings subjected to compartment fires

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Synopsis
During 1995 and 1996, a major fire test programme was conducted on a full-scale eight-storey steel-framed building. The principle aim of these tests was to investigate the actual behaviour of a steel-framed building subjected to a series of compartment (localised) fires. This paper presents the results from two of these tests, which were conducted by the Building Research Establishment. The results from the tests showed that existing fire Codes are not addressing the correct building behaviour during a fire and, as a consequence, are extremely conservative. In addition, the fire tests on the full-scale frame showed that the actual global and local structural behaviour of buildings is different, and typically far better, than that shown in standard small-scale fire tests. The paper discusses in detail the behaviour of the building observed during the fire tests, and preliminary conclusions are presented.

Introduction
When heated, steel will lose both strength and stiffness. For life safety and property protection, the use of steel within buildings will obviously need to be designed to withstand the effects of a fire. This design must ensure no additional threat, caused by possible collapse of the steel structure, to either escaping occupants or fire fighters. In addition, no disproportionate damage should occur to the building during a fire. The level of fire resistance that a structure requires is specified in terms of time in the Building Regulations'. It is dependent on the function and height of the building and, in some cases, whether sprinklers are used. In addition, maximum fire compartment sizes are specified, where the compartment boundaries are designed to contain the fire. In multistorey buildings this will typically create an area of structure that is being heated by the fire, with the structure surrounding this area remaining much cooler and thus retaining its strength and stiffness.

The most common and traditional design method of ensuring strength and stability to steel-framed buildings during a fire is to cover all exposed steel areas with a protective material. Typical types of protection material consist of boards, sprays, and intumescent paints, with the choice of material depending on cost, appearance, and durability. Specification of material thickness for a known fire resistance, time and steel section size can be obtained from individual manufacturers or from (what is commonly termed) the "Yellow Book". The required thickness is based on the principle of ensuring that the steel remains below 550°C for the specified fire resistance period. This assumes that steel members fully stressed in accordance with BS 4493 or BS 5950: Part 1 will lose their design safety margin when they reach a temperature of approximately 550°C.

Although, the philosophy of specifying protection thickness is adequate, it is extremely conservative. This is a result of the assumption that the member is uniformly heated, together with a disregard for both actual applied load levels during the fire and true material behaviour at elevated temperatures. To address this issue, research was conducted into the actual behaviour of isolated bare steel beams and columns, resulting in the first ever fire design Code, BS 5950: Part 8. This treats the occurrence of fire as an accidental limit state, with its own associated load and material partial safety factors. Similar principles were adopted in the development of EC3: Part 1.2 (which covers steel structures) and EC4: Part 1.2 (which covers composite structures). Although similar, the Eurocodes are more comprehensive and cover a much wider range of structural configurations.

The development of the design Codes provides a more solid scientific foundation for the provision of fire resistance to steel-framed structures. However, the design Codes were mainly developed from standard fire tests on isolated columns and beams. In these tests, columns are 3.0m high and beams are 4.5m long, and the failure criteria are governed largely by the size of the furnace. There is general agreement that standard tests ignore significant structural behaviour by disregarding the interaction between members. This was shown in 1990 when a fire developed in a partly completed 14-storey office block on the Broadgate development in London. Large sections of the steel frame were totally exposed at the time of the fire. Investigation following the fire allowed a back-analysis of the structure to BS 5950: Part 8. This highlighted that the Codes, although conservative, were not addressing the true behaviour, since the building was not acting as a series of individual members. The nature of a localised fire, at Broadgate, caused a zone of heated structure to expand against the surrounding cold structure. Owing to the relative greater strength and stiffness of the surrounding cold structure, additional axial forces were induced into the heated structural zone. Considering the global structural behaviour, this caused larger displacements in the horizontal and vertical members owing to the P-P-6 effect. In addition, local buckling occurred in the proximity of the connections, within the fire-affected zone. However, no signs of collapse were
design the underlying philosophy was to obtain a structure that used the minimum amount of material, was simple to manufacture, and at all stages of construction and erection reflected normal building practice, rather than specialist research procedures. The imposed load was achieved using sandbags, each weighing 11kN. This resulted in an overall applied load (dead + imposed) of 5.48kN/m²/floor.

The fire tests

A European collaborative programme of fire tests was undertaken on the test structure. The programme was coordinated jointly by BRE and British Steel. Financial support was provided by the European Coal & Steel Community and the Department of the Environment, Transport & the Regions. Other organisations were involved in the programme, i.e. the Steel Construction Institute, TNO Building & Construction Research, Centre Technique Industriel de la Construction Metallique, and the University of Sheffield. In addition to the structural experimental programme, a large risk and hazard assessment study dealing with the development of the natural fire safety concept for steel-framed buildings is currently underway.

A total of six major fire tests, which were designed to be complementary, were conducted: four tests by British Steel and two by BRE. These are summarised in Table 2, with the location of the tests (on plan) shown in Fig 2. The aims of the tests were to:
- observe, under test conditions, the behaviour of the complete building when subjected to localised fires (it was important to consider localised fires owing to the regulatory requirement of separating multistorey buildings into fire compartments)
- damage the structure in an attempt to find the limit of its inherent strength in a fire, which will enable the present design factor of safety to be quantified.

TABLE 2 – Summary of the fire tests

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Organisation conducting the test</th>
<th>Description</th>
<th>Floor area (m²)</th>
<th>Location (floor level)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>British Steel</td>
<td>One beam</td>
<td>24</td>
<td>level 7</td>
</tr>
<tr>
<td>2</td>
<td>British Steel</td>
<td>Slice across the building</td>
<td>53</td>
<td>level 4</td>
</tr>
<tr>
<td>3</td>
<td>British Steel</td>
<td>Corner compartment</td>
<td>76</td>
<td>level 2</td>
</tr>
<tr>
<td>4</td>
<td>BRE</td>
<td>Corner compartment</td>
<td>54</td>
<td>level 3</td>
</tr>
<tr>
<td>5</td>
<td>BRE</td>
<td>Large compartment</td>
<td>340</td>
<td>level 3</td>
</tr>
<tr>
<td>6</td>
<td>British Steel</td>
<td>Large compartment with office furniture</td>
<td>136</td>
<td>level 2</td>
</tr>
</tbody>
</table>
- obtain quality test data to validate, or develop further, computer models which will allow different structural and fire scenarios to be investigated, together with different size of fire compartment
- develop design guidance based on actual structural behaviour

Presented in this paper is a description of the two compartment tests carried out by BRE, with preliminary conclusions.

**BRE test 1 (corner compartment)**

A 9.0m x 6.0m compartment was constructed in the corner of the building (Fig 3), between the second and third floors. The internal compartment walls were constructed using steel stud partitions with fire-resistant board and a deflection allowance of 15mm. These were placed centrally on gridline E and slightly offset from gridline 3 (Fig 3). A full-height blockwork wall on gridline F and a 1.0m-high dado wall and double glazing on gridline 4 completed the compartment. All steel beams and beam-to-beam connections were left exposed, together with the underside of the steel trapezoidal deck. All columns were fire protected up to the underside of the floorlab. This was required to ensure that the compartment fire caused only localised damage, which will allow the rest of the building to be used after the fire. This is an extremely important factor from the insurer’s point of view. A previous test conducted by British Steel, where parts of the columns were unprotected, caused disproportionate damage to the structure, by the columns reducing in length by 200mm.

The compartment was totally enclosed, with all windows and doors closed. No additional ventilation was provided and no attempt made to artificially seal the compartment. In addition to investigating the exposed steel frame and composite floor, the behaviour of the glazing system on the development of the fire was also of interest. The glazing comprised a 9m-wide x 3m-high, 12-pane aluminium grid. Each pane consisted of 6:12:6 double-glazed sealed units 1.5m x 1.5m. To achieve the temperatures required to test the structure, the fire was designed on the basis of the glass cracking in the early stages of fire development and thus providing the maximum ventilation possible. The behaviour of the test showed that this assumption was incorrect, or at best an over-simplification.

A total of 12 timber cribs were used to give a fire load of 40kg/m². This represents the 90% fractile value for a modern office building. This means that 90% of all the office buildings surveyed had a fire load (total combustible material) less than 40kg/m². The compartment area, together with the surrounding structure in the proximity of the compartment, was extensively instrumented. This included thermocouples, displacement transducers,clinometers, and strain gauges, together with a laser system to measure thermal curvature of the full-height blockwork wall on gridline F.

After ignition the development of the fire was influenced by the lack of oxygen within the compartment, owing to the glazing remaining intact. Fig 4 shows the maximum and average recorded atmosphere temperature within the compartment. It can be seen that, after an initial temperature rise, the fire died down and continued to smoulder. This continued until the fire brigade intervened to vent the compartment by removal of a single pane of glazing, at 56 min into the test. This resulted in a small increase in temperature, followed by a decrease. Flashover did not occur until a second pane, at a lower level, was removed (at 66 min). This initiated a sharp rise in temperature that continued as the fire spread throughout the compartment (Fig 4). The maximum recorded atmosphere temperature in the centre of the compartment was 1051°C after 102 min. The maximum steel temperature of 903°C was recorded after 114 min. To put this temperature into context, the present fire design Codes suggest that the beam will ‘fail’ at 680°C. The displacement at the centre of the compartment reached a maximum of 269mm (Fig 5). Measurements taken the day after the test showed that the slab had recovered to a residual displacement of 160mm.

The unprotected edge beam (gridline 4) was observed during the test to be completely engulfed in fire (Fig 6). However, the beam reached a maximum temperature of only 680°C, with a corresponding maximum displacement of 52mm. This displacement was very small and attributed to the windposts above the fire compartment that were included in the test building. These windposts, although not incorporated directly into the design of the frame, acted in tension, providing considerable support to the weakening edge beam.

The unprotected steel beams above the compartment walls on gridlines...
E and 3 had nominal vertical displacement, resulting in the stud partition walls retaining their integrity during the fire. Fig 7 shows the stud partition intact following the fire. The beam on gridline E buckled distortionally during the test (Fig 8). This was attributed to the beam being heated on one side, together with restraint preventing thermal expansion, provided by the surrounding structure (including the stair well/core area). Although the beam on gridline E was also heated from one side only, it had nominal restraint to thermal expansion and therefore did not buckle.

Overall, the damage to the structure was extremely localised, which will allow remedial works to be carried out quickly and with little disruption to the rest of the building. All connections remained intact during both the heating and cooling phases of the fire.

Although the columns within the compartment were protected, strain gauges indicated that additional moments were induced into the columns. The values of these moments were in the range of 20–30% of the ultimate moment capacity of the columns. The structural behaviour involved in developing this increase in moment is not yet fully understood. One possible explanation is that the columns, owing to their location, had a thermal gradient through their cross-section. Restraint to this thermal gradient from the continuation of the column above and below the fire compartment could, in part, account for these induced moments. Additionally, the thermal expansion of the adjacent heated beams could have laterally displaced the columns, thus inducing restraining moments. This lateral movement would have also increased the column moment due to the P-δ effect. This presents some feasible explanations of the possible structural behaviour that may have resulted in the high induced column moments measured during the test. Further work, involving computer simulations, is under way to try to identify the predominant cause of these additional column moments.
BRE test 2 (large compartment test)

A compartment was constructed between the second and third floors, extending over the full width of the building, between gridline A and 0.5m from gridline C (Fig 9). The total floor area of the compartment was 340m². The same fire load as the previous test (40kg/m²) was placed in the compartment in the form of 42 wooden cribs. The compartment was formed by constructing a fire-resistant stud partition wall across the width of the building, and also around the vertical access shafts. Double glazing was installed on two sides of the building on gridlines 1 and 4. Unlike the previous test, it was decided to leave the middle third of the glazing open on both sides of the building. This was to allow sufficient ventilation for the fire to develop.

All steel beams, including edge beams, were left unprotected. Beam-to-beam connections were also exposed, together with the underside of the steel trapezoidal deck of the composite floor. The columns were once again protected up to the underside of the floor, to limit the structural damage to the compartment area.

In a manner similar to the previous test the ventilation conditions governed the development and severity of the fire. Rapid ignition resulted in the windows breaking during the early part of the test. This resulted in a fire scenario where the maximum temperature reached was fairly low, but the duration at which the maximum temperature occurred was much longer (Fig 10). This is a fairly severe scenario for the behaviour of the composite floor, due to the heat transfer through the concrete to the mesh reinforcement. This will reduce the strength of the reinforcement, which is the main component of resistance in these types of floor. Obviously, a fire with higher temperatures which remains constant for a long duration will be even more severe. The fire produced a maximum atmosphere temperature of 763°C and a maximum steel temperature of 691°C. This resulted in a maximum displacement of 557mm, which was measured halfway between gridlines 2 to 3 and B to C. This recovered to a residual displacement of 481mm, once the structure had cooled.

Fig 11 shows the deformed structure in the latter stages of the fire. No signs of possible collapse are evident, and overall the structure performed very well. However, there was some interesting localised behaviour that merits further discussion. Most internal beams showed signs of local buckling in the lower flange, and part of the web, in the proximity of the connections (Fig 12). It was felt that this was caused by the restraint to thermal

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**Fig 10. Maximum and average atmosphere temperatures in large compartment fire test**

**Fig 11. Large compartment test with developed fire**

**Fig 12. Typical local buckling of beams in the proximity of the connections**

**Fig 13. Typical fracture of endplate during cooling**

**Fig 14. Damage to stud partition compartment wall following the fire**
expansion and the negative moment caused by the rotational restraint from the connection. The restraint to thermal expansion was provided by the surrounding cold structure, together with differential heating through the compartment. The connections also showed signs of being subjected to high tensile forces. In the partial depth endplates the plate had fractured down one side (Fig 13) and, in one instance, the web had fractured. In the finplates (beam-to-beam connections) the bolts had sheared. The high tensile forces were induced during the cooling phase of the fire. This was caused by the steel beam, which has restraint against thermal expansion, cooling down from a plastic state. This behaviour has also been shown analytically and presented in detail elsewhere.

The stud partition wall parallel to gridline C had 15mm deflection allowance. The deflection of the slab was greater than this value and caused integrity failure of the compartment wall (Fig 14). In a fire failure of the compartment walls must be avoided and therefore the wall must be designed to accommodate the deflection of the structure or placed on gridlines (under beam positions) where the deflection will be less, as shown from the previous test.

Similar to the previous test, the strain gauges on the protected columns indicated high induced moments during the test. As explained previously, the exact structural behaviour that causes these column moments is not yet fully understood. Further work is currently being conducted to investigate the behaviour of the tested protected columns.

Discussion and utilisation of the test results

These two tests, together with the four British Steel tests, have shown comprehensively that the existing design Codes are conservative by calculating structural collapse when the beam steel temperature reaches 680°C. The maximum steel temperature in the BRE tests was 903°C, and in the British Steel tests the maximum steel temperature was above 1100°C. There was no sign of collapse in any of the tests. This raises the question of whether we reached the limit of the inherent strength of the frame, which was one of the major aims. Is it possible that the temperatures could have reached higher values? This is a question that needs to be answered by possible computer modelling.

Undoubtedly, the major contribution to the survival of the frame was the composite floor. This performed extremely well during all the tests, reinforcing the results from previous small-scale tests, which have shown that this type of floor has a good inherent fire resistance. During each test, as the beams increased in temperature and lost a large proportion of their load-carrying capacity, the composite slab supported the applied load over the fire compartment area. In the first instance this was achieved by utilising the slab’s full moment capacity, as the supporting steel beams lost their strength. As the temperatures continued to rise and the supporting beams had nominal strength, the slab utilised its tensile membrane action through its mesh reinforcement. In the case where there is no horizontal restraint (i.e. slab at an edge of a building), a compressive membrane ring will form around the area of slab which is in tensile membrane action, as shown in Fig 15.

Research work has begun to look at the contribution of tensile membrane action of the slab. It is hoped that this work will allow a fuller understanding of the mechanism and define the span (or size of fire compartment) limits.

In addition to investigating tensile membrane action, detailed computer models are being developed/validated. Fig 16 shows the prediction of the BRE corner compartment fire test, using a finite element computer model. Full comparisons between the model and test results have been presented in detail elsewhere. Once the models have been fully developed, different fire and structural scenarios can be investigated, which will enable design guides to be produced.

Conclusions

From the observations of the two tests presented in this paper, the following conclusions are drawn.

1. No collapse was evident, even though the unprotected beams reached temperatures over 900°C (temperatures over 1100°C were reached in the British Steel tests).
2. Existing fire design Codes are too conservative, predicting structural collapse at 680°C.
3. Only localised damage occurred, provided that columns were protected over their full length.
4. Non-loadbearing compartment walls placed on the gridlines (i.e. under steel beams) performed well.
5. Non-loadbearing walls placed off the gridlines showed signs of integrity failure owing to the deflection of the structure. Walls will have to be specified with an adequate deflection allowance or placed on gridlines.
6. The behaviour of the structure was different and better than that shown in standard fire tests.
7. Local buckling typically occurred in the heated steel beams in the proximity of the connections. Therefore, conservatively, the connections should be assumed to be pinned in a fire design.
8. The behaviour of the connections during cooling needs to be addressed. This may result in the need to specify more ductile connections to ensure shear capacity following a fire.
9. The composite slab was extremely beneficial to the survival of the frame. It was felt that this was due to the tensile membrane capacity of the slab, which requires further investigation.

It must be emphasised that the above conclusions apply only to buildings of the same form as the test building. Computer models are being developed to investigate different structural layouts, member sizes, compartment sizes, etc., together with different fire scenarios. Once this investigation is complete, comprehensive design guidance can be produced.

It is envisaged that most, if not all, steel beams could be left unprotect ed. The saving from not using passive fire resistance in terms of material cost and time required to fix/apply the protection could be used to provide active fire measures (such as sprinklers). This will inevitably create overall safer buildings.

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References

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