NEW RESEARCH IN THE UNDERSTANDING OF STEEL FRAMES IN FIRE AND THE DEVELOPMENT OF NEW FIRE ENGINEERED SOLUTIONS APPLICABLE TO TALL BUILDINGS

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Abstract

A major fire research programme has been carried out on an 8-storey composite steel frame involving natural fires. The test arrangements and results are described and the important factors that control the structural deformation processes are discussed. One of the main conclusions from this work is that modern composite steel frames are extremely robust in fire in the event of loss in strength of individual steel members. Some of the numerical modelling carried out is described and the resulting design guidance that is now emerging applicable to tall buildings is presented. Comments are also made with regard to the behaviour of bolts and welds at connections.

A new structural solution is introduced consisting of composite steel and concrete panels. These can be used for constructing escape and fire fighting shafts in tall buildings that offer economies in design and resistance to both impact and fire.

Keywords: Fire, Composite, Structures, Stability, Connections, Bi-steel

1. Introduction

For many years, the performance of structural frames in buildings has been based upon single element tests carried out in a furnace in which the temperature rise follows an international heating curve ISO 834. Failure of the member is deemed to occur when either a limit of deflection or rate of deflection is achieved. While these test methods have in the past, served the industry well in providing a basis for prescriptive approaches set by individual National Regulatory Authorities, the periods of fire resistance set are often disaster lead. They do not necessarily reflect an understanding of the true risk to a structure or indeed, an understanding of the level of safety that the measures introduced have against the perceived level of risk.

During the last few years significant advances have been made in understanding the behaviour of steel composite frames in fire. In particular, there is now recognition of the beneficial effects of the load redistribution paths that are provided in a real structure through a metal deck composite floor system when one or more members loose their load bearing capacity. In addition, the performance of connections in providing stiffness to the steel frame is seen as a contributory factor in providing structural stability under real fire conditions. From an understanding of the composite interaction between steel and concrete, new structural systems have been developed that also offer significant advantages in terms of cost, structural integrity and life safety under both impact and fire.

The purpose of this paper is to review some of the most recent advances in our understanding of steel composite structures that are relevant to the design of tall buildings.

2. Cardington, UK Fire Test Programme on an 8-Storey Composite Steel Frame

Corus recently completed a series of fire tests conducted on a modern steel composite 8-storey steel frame structure built within the Cardington fire test facility in the UK. The purpose of the research was to understand the extent of restraint and redundancy in modern composite steel frames when they are subject to fire attack. The end product to this research was to identify the critical elements in structures exposed to fire and to develop design guidance for use by the practising engineer in order to specify the fire resistance requirements adequately and economically.
The structure was designed to meet current the UK National design codes incorporating three stiff cores (a central lift shaft and two stair wells at either end of the building), with primary partial depth end plate and secondary fin plate connections.

Composite action was achieved by shear studs welded through trapezoidal steel decking onto both the primary and secondary beams. The slab was cast using lightweight concrete with an in-situ density of 1900kg/m³ to provide a maximum floor thickness of 130mm. This incorporated an A142 anti-cracking mesh.

In order to rationalize on sizes, standardize on connection details and so reduce fabrication and erection costs, only four beam sections (254UB trimmers, 305UB ribs, 356UB and 610UB spine members) and three column sections (305UC x 198 & 118kg/m and 254UC x 89kg/m) were used.

2.1 Experimental Programme

Six tests were conducted on the steel frame. Two of the tests were carried out by the Building Research Establishment (BRE) and were sponsored by the UK Government. Four tests were carried out by the Corus Fire Engineering research team and these were jointly sponsored by Corus Group and the European Coal & Steel Community (ECSC). TNO - Netherlands and CTICM - France were also project partners. A floor plan showing the location of the tests within the steel frame is illustrated in Fig. 1.

![Floor Plan Showing the Test Locations](image)

The tests were designed to investigate different aspects of structural behaviour and increased in complexity as the work progressed. Those carried out by Corus Fire Engineering are generically referred to as:

(i) 1D - Restrained Beam
(ii) 2D - Plane Frame
(iii) 3D - Corner
(iv) Office Fire - Demonstration

Throughout the programme, the test areas and the surrounding frame were extensively instrumented to measure both physical and engineering parameters. Up to 600 separate pieces of instrumentation were installed in each test.

2.1.1 Test 1: 1D - Restrained Beam

The first test was carried out on a single beam and the surrounding floor slab. It was designed to investigate the structural deformation when a single beam is heated and is restrained by the surrounding steel and composite floor slab spanning in two directions.

A furnace was constructed around the beam and surrounding floor slab and this was heated at a rate of between 3-10°C/min until temperatures of 800-900°C were attained through the section's profile. At this point, the mid-span deflection had attained 230mm (L/35). For comparison, in the UK Fire Limit State Design Code BS 5950: Part 8 (1990), a single element subjected to the same load ratio would
have attained the limiting deflection criterion of L/30 in the Standard Fire Resistance Test BS476: Part 20 (1987), BS476: Part 21 (1987), at a lower flange temperature of less than 700°C.

After the test, examination of the damage revealed that the beam had buckled at both ends just inside the furnace. There was also evidence of buckling in the web and bottom flange at the ends of the beam as it expanded and rotated against the web of the column section, Fig. 2. During cooling, large tensile forces were generated along the length of the test beam as thermal contraction of the deformed member was restrained by the colder surrounding frame. This resulted in shear failures on one side of the partial depth end plate at both beam/column connections.

![Fig.2 Deformation at the End of the Test Beam](image)

### 2.1.1 Test 2: 2D - Plane Frame

The second test was designed to evaluate the behaviour of a series of beams and columns supporting the 4th floor by taking a 2D slice across the full width of the building. An important aspect to the test was to determine how important fire protection should be extended around the connections when the columns would normally be protected. For this reason all the columns were lightly protected to a height of 200mm below the connections. The beams as well as the beam to column connections remained totally exposed.

A furnace 21m long x 4m high was constructed to form a corridor across the full width of the building. The structure was heated over a period of 2.5 hours during which the lower flange in the beams attained temperatures of around 800°C with a maximum temperature of 850°C being recorded. The main 610mm deep primary beam deflected 293mm over a span of 9m (Span/31), which recovered to 237 mm deflection when the structure cooled back to ambient temperature.

Temperatures in the exposed portion of the columns just below the connections attained 750°C and while they appeared straight over the majority of their length, the column heads had grossly distorted and in effect had been squashed 180mm of the original 200mm clearance between the top of the fire protection and the underside of the beam. In contrast, the columns the columns at either end of the frame were only slightly distorted. It was clear from a close examination of the beam to column connections the expansion and deformation of the beam, in particular around the lower flange, had occurred in a similar manner to that reported in test 1, Figure 2.

### 2.1.1 Test 3: 3D - Corner Test

The objective of the third test was to evaluate the behaviour of a complete composite floor system and in particular the importance of membrane action in the floor slab. A compartment with a floor area of 80m² was built on level 1. A fire loading of 45kg of wood/m² made up of wooden cribs was used as the fuel and coupled with a ventilation opening factor of 0.34m²/² was designed to develop steel temperatures around 1000°C. The measured heat output was 19MW.

Based upon the results of the earlier tests, the columns were fully protected including the primary beam to column connections. However, all the floor beams including the beam-to-beam connections, remained totally exposed.

During the fire, the 356mm primary beam achieved 864°C and the secondary beams achieved a maximum temperature of 1021°C in the lower flange. Deflections at mid-span of the primary beam and secondary beams varied from 164mm over 6m span (L/37) to 428mm over 9m span (L/21). At these temperatures, structural steel has less than 10% of its ambient temperature strength.
As reported in the earlier tests failures did occur at several connections and again this was identified as occurring during the cooling down process. The discoloration on one of the fractured end plates indicated failure occurred at 310°C and in one of the bolts, there was no discoloration at all thereby indicating a much lower failure temperature.

2.1.1 Test 4: Office Fire - Demonstration Test

The purpose of the fourth test in the programme was to demonstrate some of the important conclusions reached in the earlier studies in a more realistic scenario while at the same time evaluating other aspects of structural behaviour not previously addressed.

A compartment up to 18m wide and 10m deep was built on the first floor of the building to represent a modern open plan office. This was fitted out as a series of work stations using modern furnishings, computers and filing systems. A detailed analysis was undertaken to quantify the total combustible content and this was found to be equivalent to 45.6kg of wood/m² of floor area. This quantity of combustible material represents in excess of the 95% fractile for office fire loadings in accordance with the UK Fire Engineering Design Code (1987).

For the test, all the columns were protected including the main beam to column connections. However, the primary and secondary floor beams remained totally exposed including the beam-to-beam connections. Fig. 3 shows a general view of the office layout prior to ignition, during and after the test.

![Fig. 3 Before, During and After the Office Fire](image)

The fire was started in one corner of the room and then was allowed to progress naturally without any intervention. Within 10 minutes of ignition, a temperature of 900°C was achieved and a maximum temperature of 1213°C was attained after 33 minutes. At its peak, the heat output was 58MW. The temperatures in the exposed beams attained between 1000°C and 1100°C. Thermal cycles of the fire and one of the steel members is illustrated in Fig. 4.

![Fig. 4 Thermal Cycles of the Atmosphere and Unprotected Beam](image)

The entire contents of the office were consumed in the fire. The primary and secondary floor beams deflected up to 642mm over a 9m span (L/14) but despite the excessive deflections and weakening of the steel, the composite floor system remained in place. Above the test floor, it was clear that
compartmentment had been breached but the evidence indicated this had only occurred during the cooling down stage.

This last test clearly demonstrated the resilience of modern composite floors to fire attack even when the accepted critical temperatures of steel members are grossly exceeded.

2.2 Numerical Modelling of Steel Framed Buildings Subject to Fire and Design Guidance

The results of the fire test programme clearly show that composite steel frames behave considerably better than by considering the fire resistance performance of individual members in isolation. However, in order to capitalise on this performance in the design of composite structures it is fundamental that the models developed can replicate all of the important structural effects.

This has been studied at two levels:

(i) Local behaviour of the individual structural members.
(ii) Global behaviour of the overall framework or sub-frame system

2.2.1. Local Behaviour

The behaviour of a single member in fire is governed by three main factors:

(a) Temperature distribution in the member.
(b) The load carried by the member.
(c) The degree of restraint provided by the surrounding frame.

An example of the type and sophistication of the modelling approach to single elements is illustrated in Fig. 5. In this example, which is taken from the first test on the restrained beam, the local deformation characteristics have been identified in terms of temperature and changes in strain output from the gauges installed around the connections. By analysing each of thermo/physical events that took place during the test, an understanding of the whole deformation process was possible. The resulting analysis was able to mirror the exact sequence of events and most importantly at the correct temperatures. The modelled behaviour established using finite element analysis can be compared against the observed deformation shown previously in Fig. 2.

![Graph showing strain gauge outputs and modelled behavior of the first test.](image)

**Fig.5 Strain Gauge Outputs and Modelled Behavior of the First Test**

Currently, the deformation processes of the tests have been successfully replicated and these now form the basis of developing sophisticated models to describe the overall behaviour of the structural frame. By conducting a series of parametric studies, it is expected simplified design guidance will be produced which can be applied to buildings for a range of structural layouts. However, in the meantime, a global approach can be taken to some aspects of the structural response and this is presented in the following section.
2.2.1. Global Behaviour and Design Guidance

Based upon the results of the Cardington tests and work carried out elsewhere, the behaviour of the composite floor slab has been studied in detail. It is clear that although for ambient temperature the composite floor slab is normally considered to span in one direction, its behaviour in fire is quite different. Initially, during a fire, when the deflections in the slab are still small, compressive membrane action occurs after yielding and is dependent upon the restraint provided at the edges. As the deflection increases tensile membrane action occurs which can result in fracture of the reinforcement. The role played by the reinforcement is clearly critical and indicates that a mesh with greater ductility would be beneficial.

From this understanding, design guidance has been published by the Steel Construction Institute (2000), which considers the fire resistance of composite action and protection requirements such that it is possible to design out the fire protection of internal secondary beams. In the design, the floor layout is considered as a series of zones and the requirements for the mesh reinforcement is a key feature of the design process.

The guidance that is now available and is applicable to floor beams designed compositely with the floor slab covers:

(i) Braced, non-sway frames in which the moment resistance of the connections is ignored.

(ii) The composite floor slab comprises steel decking, reinforcing mesh and normal or lightweight concrete with the decking being either trapezoidal or re-entrant profile.

The guidance does not cover fabricated beams or unprotected beams with large multiple openings or pre-cast slabs.

2.3 Connection Behaviour

The behaviour of connections in composite frames has recently been placed under scrutiny and their performance has been subject to criticism as a weakness in steel frameworks.

In the Cardington experiments, shear and tensile failures in the bolts and welds at the beam to beam and beam to column connections were not uncommon. However, when these failures were examined in some detail, it was clear that they occurred (without exception) during the cooling phase of the fire. Visual inspection of the weld failures in particular exhibited characteristic tempering colours on the fracture surfaces and these could be associated with the various stages in the oxidation process. Based upon the extent of tinting, the actual temperatures at which the failures occurred could be identified to within 20°C. Since this type of surface discolouration can only originate from an exposed (fracture) surface, they signify the highest temperature of the component when the failure occurred. In addition to the visual observations, the actual failure event was often identified through abrupt changes in the electrical outputs from strain gauges fitted around that particular part of the frame.

From the thermo-mechanical analysis/inspection described above, it is clear that during the cooling process very high tensile forces were generated at the connections due to the thermal contraction of the deformed/deflected beam. However in all cases, due to the continuity of the composite floor slab provided through the metal decking and reinforcement, at no stage was the structural stability of the frame was brought into question.

Until recently the design of connections (bolts and welds) at the fire limit state has been limited by the lack of information on the elevated temperature material properties of these types of components. To some extent, this deficiency in our knowledge has been partly rectified and work reported by Kirby (1995) and Latham and Kirby (1998), has been taken into account in producing European design rules (prEN 1993-1-2, 2002) for both welds and high strength bolts at temperatures up to 1000°C.
3. Development of Composite Panels, Bi-steel

During the last few years, a new structural system has been developed and patented referred to as Bi-steel.

Essentially this comprises of two outer steel plates that are separated by steel bars friction welded to the inner faces. The gap between the plates is then filled with concrete or other materials to suit the particular application. The overall thickness of the panel can vary between 200mm and 700mm and these can be bolted or welded together to create various forms of construction. This type of system often provides economic advantages in terms of weight, overall thickness and speed of construction against conventional steel reinforced concrete solutions. Further details are presented by Dixon and Bowerman (2003).

In the context of impact and life safety in fire, one of the advantages of Bi-steel is that under impact loading any failure of the concrete between the two steel plates is held in place. The system has therefore potential applications in the construction of shaft walls for escape stairs or fire fighting shafts where both impact and resistance to fire is required. Figure 6 shows a schematic shaft wall construction using Bi-steel panels in which openings have been incorporated at each floor level.

![Fig.6 Schematic Representation of a Building Shaft Constructed using Bi-steel Panels](image)

In tall buildings, the structural fire resistance requirements are usually demanding particularly where shafts are constructed for safe evacuation and these are also intended to be used for seek and rescue operations by fire fighters. Fire resistance tests carried out on Bi-steel panels have shown they have already have some inherent fire resistance depending upon the thickness of the panel. Although fire protection would be specified to achieve the equivalent of 2 and 4 hours fire resistance under hydrocarbon and cellulose heating regimes, only a thin layer would normally be required to achieve these periods of stability, integrity and insulation. In such situations, it is vital that where mechanical impact is likely, the protection system is able to withstand the forces generated and still provide the necessary levels of fire performance. Based upon current technologies, an epoxy based intumescent coating is the most appropriate fire protection system in thicknesses of around 1/4" (6mm).

Conclusions

During the last few years research and development carried out within Corus Group has played a major contribution in the understanding of composite structures under fire and impact. While it is clear that our understanding and the development of cost-effective solutions has much further to go into the design of 'safe' buildings, progress is being made towards a performance based approach to life safety in tall buildings.
References


