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Title : Large-scale fire test of unprotected cellular beam acting in membrane action

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ABSTRACT

This paper describes a full scale fire test performed recently on a composite floor for analysing the possibility of tensile membrane action to develop when the unprotected steel beams in the central part of the floor are made of cellular beams.

The natural fire was created by a wood crib fire load of 700 MJ/m² and the 9 x 15 m floor survived the fire that peaked at 1000°C and lasted for about 90 minutes.

Blind predictions of the air temperature development by the software OZone and of the structural behaviour by the software SAFIR which proved quite satisfactory are also described.

Keywords : Composite Structures, Fire engineering, Research & development, Steel Structures

INTRODUCTION

As spans become longer, steel framed buildings become then more competitive compared with reinforced concrete framed buildings. For maximum economy, steel beams should be designed to act compositely with the floor slab. The increased use of long span composite beams leads to large open plan offices with minimal columns. However, as the span increases, the beam depth will also increase which, in turn, can lead to increased storey heights. The use of cellular beams (CB) largely overcomes this problem because ducts, pipes and other services can pass through the openings in the web. Also, as CB are constructed from hot rolled sections, the increased section depth results in added strength without additional material and thus tends to reduce the total weight of steelwork. Efficient assessment of structures in fire conditions is becoming more and more relevant and is covered by the use of numerical models. However, numerical models are based on small scale tests and experience. To date, no rigorous research into the performance of cellular beams in fire has taken place. The design assumptions are still largely based on the performance of solid web beams in standard fire tests.

A large scale composite floor using cellular beams connected to composite slabs was tested under a natural fire (Figure 1). The two central secondary beams were left unprotected. As cellular beams behave in a very different way compared to traditional steel beams in fire conditions, the test also provided unique experimental data on the performance of the cellular beams acting in membrane action. All the beam sections (protected and unprotected) and the slab were instrumented in order to measure the evolution of temperatures and displacements during the fire.

Figure 1. Inside view of the compartment before the test

The fire test was conducted on the 27th of February 2010 by the University of Ulster (Figure 2). The information recorded during the test will be used to validate the natural fire safety concept and provide design rules and guidance for protected and unprotected cellular beams. The work is supported by the Research Fund for Coal and Steel and six partners are involved in this project.

Figure 2 Fire test and structural elements after the fire

The compartment covers an area of 15 by 9 m with a floor to soffit distance of 3m, such as would be found near the central zone of any office building. The surrounding walls of the compartment were made of normal weight concrete block works with three 3 x 1.5 m openings in the front wall. The surrounding walls were not fixed to the composite floor at the top which allowed vertical movement of the floor without interaction with the walls. All the columns and solid beams on the opening side were protected for a standard fire of two hours using fibre boards of 20 mm. The surrounding cellular beams were also protected using ceramic fibre for fire duration of two hours.

STRUCTURE

The slab is made of 51 mm deep profile of the Kingspan Multideck 50 type with a concrete cover of 69 mm on the profile, which makes a total depth of 120 mm. A steel mesh of 10 mm with a spacing of 200 mm in each direction made of S500 steel was used as reinforcement. It was located at a vertical distance of 40 mm above the steel sheets. The slab was fixed on all steel beams by means of steel studs welded on the upper flanges (full connection). All connections from secondary beams to main beams and from beams to columns are pinned connections. Horizontal bracing was provided in 4 positions leaving the slab completely free of external horizontal restraint.

DESIGN LOADS

The loads applied on the slab are those which are commonly used in the design of office buildings, see Table 1.

description	Characteristics KN/m^2	Fire Factor	Design Load KN/m^2
Partition	1.0	1.0	1.0
Services & Finishes	0.5	1.0	0.5
Live Load	3.5	0.5	1.75
		Total	3.25

Table 1. Design Loads

The applied load of 3.25 kN/m^2 was achieved using 44 sandbags of 1 tonne evenly positioned over the floor plate, as shown in Figure 3a. The self weight of the slab of 120 mm thickness is about 2.90

KN/m^2. The safety factor for live load has been taken at 0.5. It corresponds to the ψ_1 of EN 1991-1-2 which is the maximum value that is recommended in the Eurocodes. Taking this upper limit increases the utilisation factor of the structure in fire conditions.

FIRE LOAD

Assuming the design for an office, the fire load density would be 511 MJ/m^2 according to Table E.2 of EN 1991-1-2 [EN1991-1-2, Fire design]. However for this test, the fire load was increased by using 45 standard (1m x 1m x 0.5m high) wood cribs, comprising 50 mm x 50 mm x 1000 mm wooden battens, positioned evenly around the compartment (Figure 3b), yielding a fire load of 40 kg of wood per square metre of ground area. The wood density provided was 510 kg/m^3 with a calorific value of 17.5 MJ/kg for wood, which corresponds finally to a fire load of 700 MJ/m^2. This is consistent for multi-storey office accommodation [CIB W14 Workshop Report, Fire Safety Journal 1986] and allows a direct comparison with previous test carried out on the steel building at Cardington [Bailey C.G. et all, April 1999]. The figure is well established from the statistical data and a number of tests have been carried out considering the quantity of fire load as the variable parameter [British Steel Technical, Fire Research Station Collaborative Project].

Figure 3. a) Vertical static load b) Wooden cribs used for the fire load

METHOD OF IGNITION AND TEMPERATURE IN THE COMPARTMENT

The fire was started from a single ignition source (Figure 4). After 5 minutes, the firemen decided to start two additional ignitions sources in different places and the rest of cribs were left to ignite naturally. Each crib was connected to its neighbours by mild steel channel section with porous fibre board laid into the channels and, approximately 30 minutes before ignition, some 20 litres of paraffin was poured into channel.

Figure 4. Ignition and fully engulfed fire

A blind prediction of the temperature development was made using the software OZone [Cadorin J.-F. & Franssen J.-M., Fire Safety Journal 2003 ; Cadorin J.-F. et all, Fire Safety Journal 2003] with the following hypotheses:

- The fire load density: 570 MJ/m²
- Combustion model: extended fire duration
- Fuel height: 0.5 m
- RHR_f: 1250 kW/m²
- Combustion heat of fuel: 17.5 MJ/kg
- Fire growth: medium
- Combustion efficiency: 0.8

As the fire test was conducted with a fire load of 700 MJ/m²,.a second calculation was performed with this fire load without changing other parameters. Figure 5 shows the comparison between the measured temperatures in the compartment and the OZone predictions:

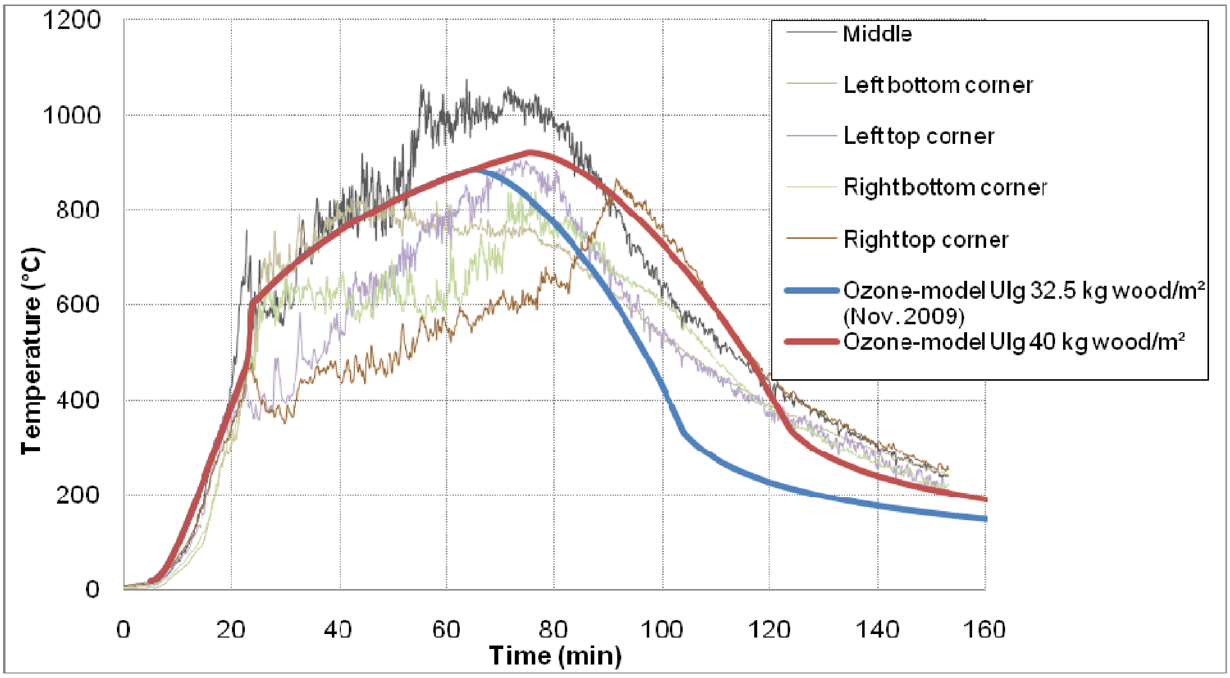
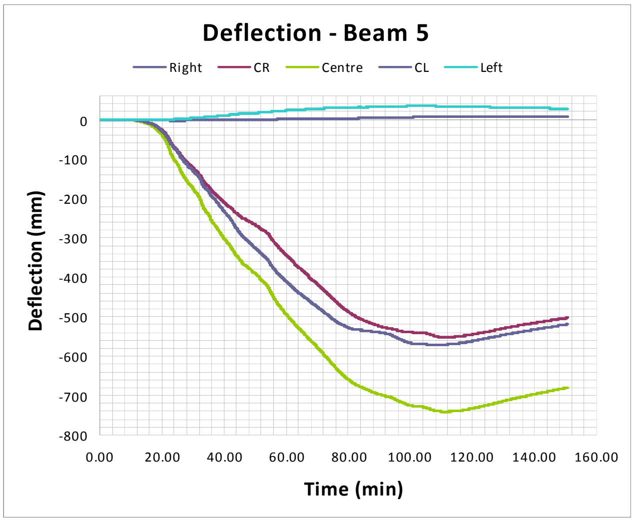


Figure 5. Measured temperature in the compartment Vs OZone prediction

LONG CELLULAR BEAM BEHAVIOUR

Under fire conditions, the deflection in the steel beam is the result of two causes: the thermal bowing and the mechanical deflection. The mechanical deflection is the increase in deflection under constant load due to reduced steel strength and stiffness with increasing temperatures. It is expected that at low temperatures (less than 500°C), the beam deflection is controlled essentially by thermal bowing. At higher temperatures, mechanical deflection dominates and the beam deflection increases at a faster rate (Figure 6) with a rise in the beam temperature (Figure 7). The unprotected cellular beams were not able anymore to support the slab in bending behaviour. Due to local buckling all along the web, see Figure 6, only the top tee of the beam was able to participate to the bearing capacity of the system.



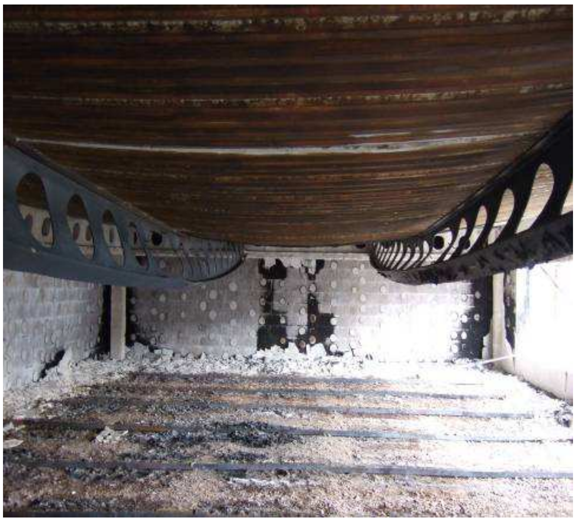
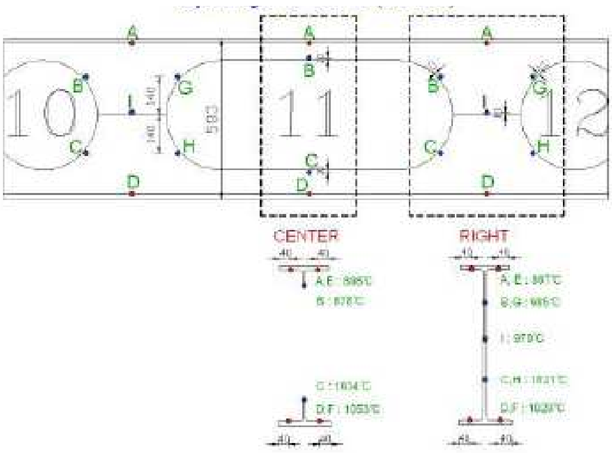


Figure 6. Deflection of the unprotected beam and slab

Openings 10 TO 12 (B5 -Z3)



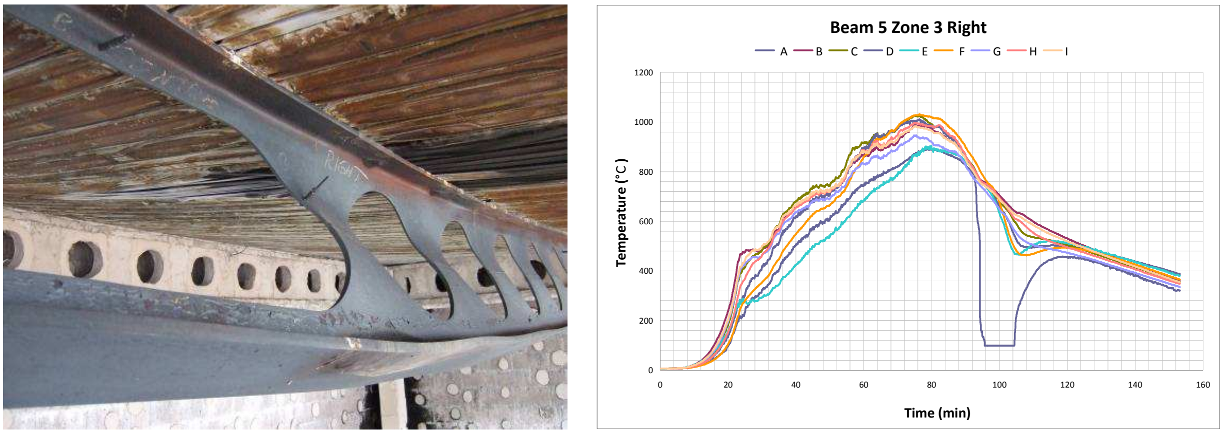


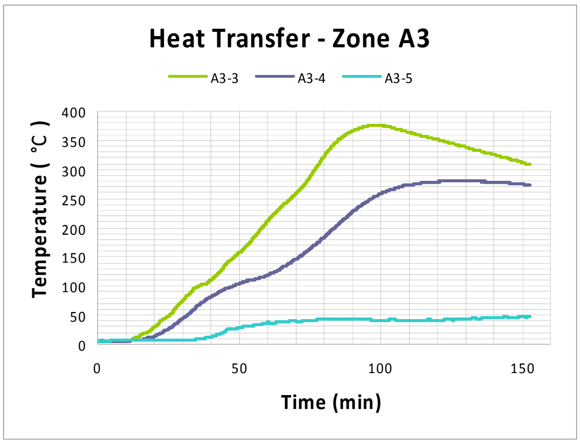
Figure 7. a) web post buckling , lateral and torsional effect,
b) temperature distribution at the steel cross section

FLEXURAL STRENGTH OF THE COMPOSITE SLAB

The concrete slab had a nominal thickness of 120 mm and was constructed using normal-weight concrete. The average cube strength was 54.8 N/mm2 at 28 days. The slab was exposed in an external environment and, at the time of the test, the measured moisture content of the concrete slab was 6.4% by weight. The slab reinforcement consisted of welded wire mesh reinforcement A393 (10mm diameter ribbed bars at 200mm centres) having nominal yield strength of 500N/mm^2. Full interaction between the slab and beam was ensured in all specimens by the use of a high density of shear connectors of 19 mm diameter studs at height 95 mm. The shear studs have been equally distributed in one row with spacing of 150 mm over the beam length. A trapezoidal steel deck with a thickness of 1.0 mm was used as sheeting.

Recorded results show very high temperatures in the steel decking, reaching the maximum of about 1100°C. The steel decking was also observed to have debonded from the concrete slab in most areas. Thus it may be assumed that the steel decking contributed very little to the slab strength at the maximum fire severity. In cold conditions, the tensile forces of the slab in bending behaviour were taken by the steel sheet alone. Due to the high temperature of the steel decking in fire conditions, no tensile stresses can be taken anymore by this steel and no additional rebars were added in the ribs to replace the steel section of the sheet. So the bending resistance of the slab in fire condition was really limited.

It was clear from the test that the structure survived to the fire and the only possible physical behaviour is that the membrane action occurred in the floor plate. This supports the current design approaches [Bailey C.G. & Moore D.B., June 2000 - Bailey C G & Toh W.S., 2007] which utilise this mode of behaviour to allow a significant number of steel beams to be left unprotected.



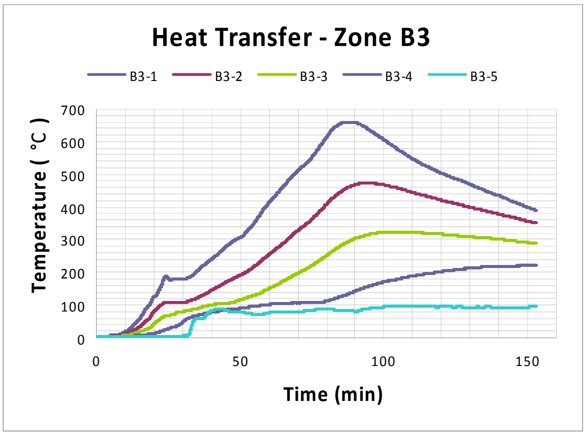


Figure 8. Temperatures at the slab decking

SAFIR FINITE ELEMENT PREDICTION

A finite element model was built in the SAFIR software [Franssen J.-M., 2005]. This model was made blind before the test in order to predict the behaviour of the structure. Figure 9 shows the numerical modelling with different types of elements.

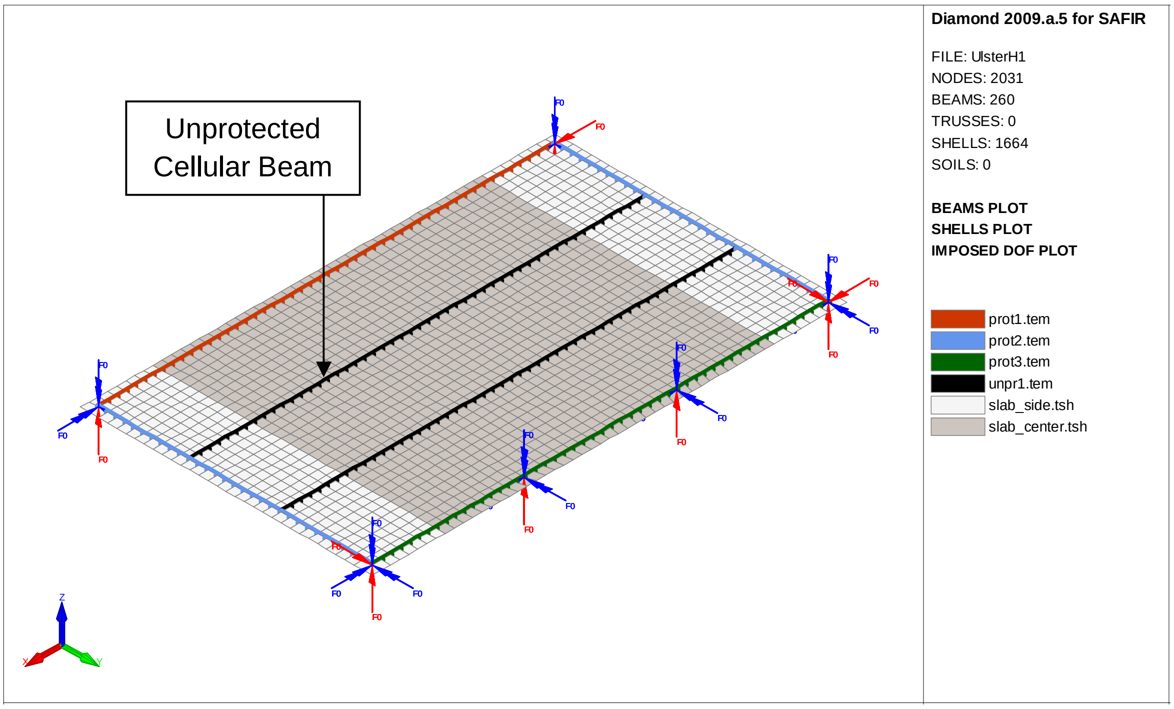
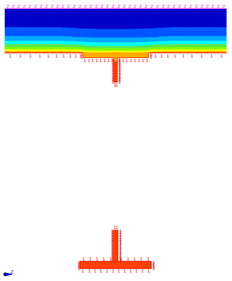


Figure 9. Finite element model built in SAFIR

The unprotected cellular beams were simulated using BEAM finite Element which does not allow taking into account the web post buckling instabilities. So, the simulation was run twice, once with cellular beams modelled as the double tee section (Figure 10a) and once as only the upper tee section

(Figure 10b). Theses two models allow to simulated the cellular beams before and after the web instabilities.



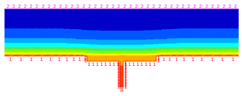


Figure 10. a) double tee section b) upper tee section

The lower curve on the Figure 11 is obtained by modelling only the upper tee of the unprotected beams, what is justified by the fact that web post buckling will appear in these sections and will prevent the bottom tee from playing any structural function. In this case the deflection at room temperature has no physical signification since the real contribution of the secondary beam is largely underestimated. But in fire situation, the results are interesting. For example, it can be observed that the deflection does not decrease when the temperature decreases, because the steel profiles do not recover their stiffness. This model can be considered as a reasonable model for a simulation of such type of floor system in the fire situation since the cellular beams, after the web post buckling, will probably not be able to recover their initial stiffness when the temperature decreases.

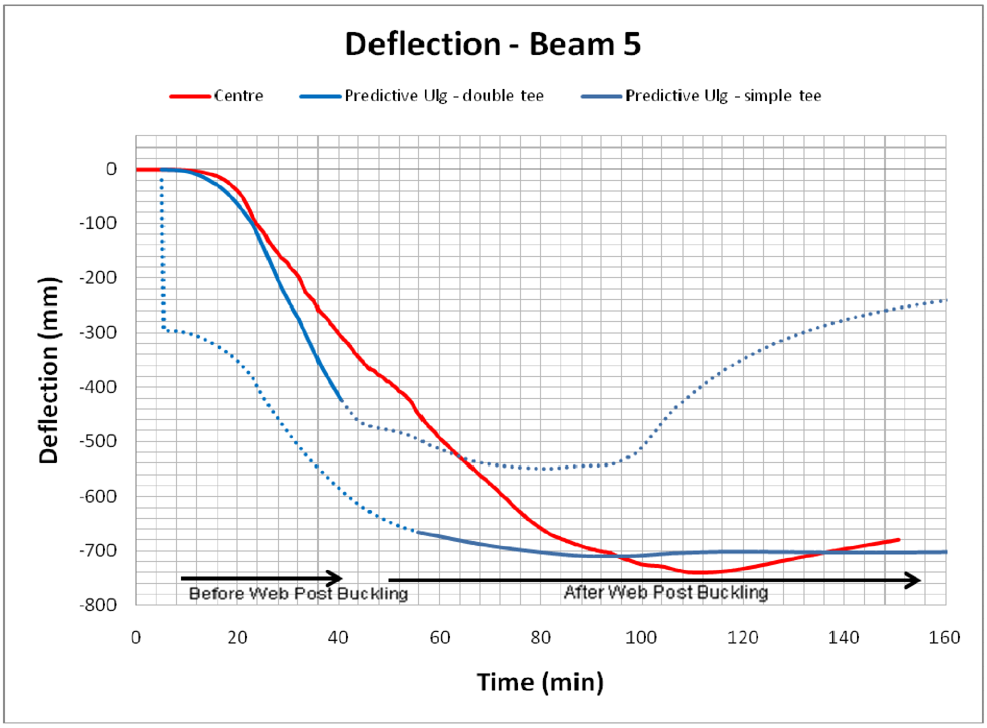


Figure 11. Influence of the model of the unprotected beams and comparison with test results

Figure 11 shows a good correlation between the FEM model and the real behaviour of the test. Of course, some parameters of the finite element model can be adapted in order to fit with the real properties of the material used during the test, the real measured temperatures in the element, etc.… But it already gives some confidence that this model is capable of predicting the fire behaviour of such type of floor system with a satisfying level of accuracy.

It would also be possible to model the steel cellular beams in detail with shell elements, but such model would be too large for practical applications.

CONCLUSION

This fire test provided a unique opportunity to study the behaviour of long cellular steel beams in a complete compartment office in building structure under realistic fire conditions. The test was very successful, fire was more intense and of longer duration that assumed in the initial studies yet the structure performed as predicted. As shown in the numerical simulations, it appeared clearly during the test that the fact to use cellular beams to support the composite slab does not jeopardise the tensile membrane action that develops in the slab in a fire situation.

The OZone model provides a rather good estimation of the fire development, provided that the correct amount of fire load is introduced.

The SAFIR structural model was capable of predicting with an acceptable level of accuracy the complex behaviour of cellular beams acting in membrane action.

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