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A Structural Fire Engineering Prediction for the Veselí Fire Tests, 2011

by

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A Structural Fire Engineering Prediction for the Veselí Fire Tests, 2011

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Abstract
Fire hazards and full-scale structural tests have provided evidence that the beam-column connections of building frames are the weakest structural elements, which are vulnerable to fracture in fire. Connection fractures may lead to extensive damage or even progressive collapse. However, current design methods for connections are solely based on ambient-temperature behaviour, the additional forces and rotations generated in fire are not taken into account. The Structural Fire Engineering Research Group of the University of Sheffield is involved in a European-collaborative project which concerns the behaviour and robustness in fire of practical connections to composite columns. This includes two natural fire tests in a full-scale composite structure in Veselí, the Czech Republic. The Sheffield team was responsible for predicting the structural behaviour in the tests before they were conducted. This assessment was conducted using the specialist structural fire engineering FEA program Vulcan. This paper reports the results of this predictive analysis.

Keywords: Modelling, Fire Test, Connection, Composite Columns, Robustness

1. INTRODUCTION
When a fire occurs in a building the internal forces in the joints change substantially during the course of the fire, even though the external forces applied to the structure remain unchanged. This results mainly from restraint to thermal deformations and degradation of the mechanical properties of the building materials at high temperature. These phenomena were observed in both full-scale tests [1] and actual fires [2, 3]. Because current design methods for connections are solely based on ambient-temperature behaviour, the additional forces and rotations generated in fire are not taken into account. If, at any stage of fire exposure, a connection does not have sufficient resistance to accommodate the large rotations and co-existent forces, a local failure (such as plate tearing and bearing failure; bolt tensile breakage, shear, pull-out, thread stripping) will occur. This may lead to extensive damage or progressive failure of the structure. Therefore, the way the joints perform in a building fire will be critical to whether it would be able to survive the fire attack.

The Structural Fire Engineering Research Group of the University of Sheffield was a participant in the European-funded project COMPFIRE [4], which was a collaboration between research teams at universities in Manchester, Coimbra, Luleå and Prague, Desmo Ltd in the Czech Republic, and TATA Steel RD&T. The objective of this project is to investigate the behaviour and robustness in fire of practical connections between steel beams (both composite and non-composite) and two types of composite columns - concrete-filled tubular (CFT) and partially-encased H-section columns. Two natural fire tests on a full-scale composite structure took place in Veselí, in the Czech Republic [5]. One task of the Sheffield research team had been to predict the structural behaviour of the tests before they were conducted. The assessment was conducted using the specialist structural fire engineering FEA program *Vulcan,* and this paper reports the results of this predictive analysis.

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2. TEST DESCRIPTION

The objectives of the Veselí tests were to provide experimental data on the response of composite frames to a natural fire, and to demonstrate the impact of improved detailing of joints on structural robustness in fire. The tests were conducted on the 6th and 15th of September 2011. They were designed and carried out by the Prague team of COMPFIRE (Department of Steel and Timber Structures of the Czech Technical University) and the Sheffield team checked the design of the connections. Two fire tests were performed on the test structure in sequence, one on each storey. During the first test (hereafter referred to as Test 1), the upper storey was heated without mechanical loading applied on the slab above. The peak heat release rate predicted by Fire Dynamics Simulator (FDS) simulation was approximately 15MW. This test did not aim to cause failure; the objectives were to observe the heat transfer in the compartment and to measure the temperature distributions in the structure, particularly in the beam-to-column connections. The second test (Test 2) was performed in the lower storey, which was subject to both mechanical and fire loading, with the upper floor cool. It aimed to collect temperature data and to demonstrate the robustness in fire of the particular types of joints investigated in this project. Failure of the connections had been expected. In both tests, the temperatures of gas and structural elements, deformations of slabs as well as the forces on connections were measured.

The test was set up as a 9m high, 10.4m x 13.4m two-storey office building structure as shown in Figures 1 and 2. The floor system consisted of 120mm thick composite slabs with trapezoidal decking and Grade C30 concrete. The slabs acted compositely with the steel beams with TRW Nelson 19mm diameter and 100mm height shear connectors in each rib. The slabs were reinforced with a plain bar mesh, providing a steel area of 196mm^2/m in each direction, situated 20mm below the top of the slab. There was also a ϕ8mm reinforcement bar in each rib, with 20mm cover from the bottom of the slab.

The edge beams were all IPE 240, while IPE 270 beams were used for the 9m span interior beams and IPE 220, IPE 200 and IPE 180 were used for the shorter-span ones. The floor arrangement was supported by five circular CFT columns (TR 245/8 filled with Grade C30 concrete) and four HEB 200 steel columns at the corners. All columns and beams were of Grade S355 steel. Two types of connections, reverse-channel and fin-plate, were adopted. The horizontal stiffness of the building was provided by two sets of cross bracing of tubular section in each direction of each floor. The columns, edge beams, bracings and connections in the lower floor were fire-protected using fire spray PROMASPRAY F250 (a mixture of mineral fibres and cement binder) to a standard resistance R60. All the other elements were unprotected. The cladding walls, which were composed of liner trays, mineral wool and external corrugated sheets, formed a fire compartment on each floor.

The applied mechanical load was designed to represent that of a typical office building. The imposed load of 3.0kN/m^2 on the slab was generated by bags filled with gravel and recycled concrete. The self-weight of the floor system (including the slabs and floorings) was 2.85kN/m^2 and that of the partitions was 0.5kN/m^2. The fire load was generated by timber cribs of size 50mm x 50mm x 1000mm to simulate a natural fire in a regular office building. A 2m x 6m unglazed opening in the front wall of each floor provided ventilation to the fire. More details of the test setup can be found in Wald [5].

3. PRE-TEST MODELLING USING VULCAN

This section reports the pre-test predictions of the structural response of the loaded test (Test 2). The modelling was conducted using the structural fire engineering FEA program *Vulcan* [6].

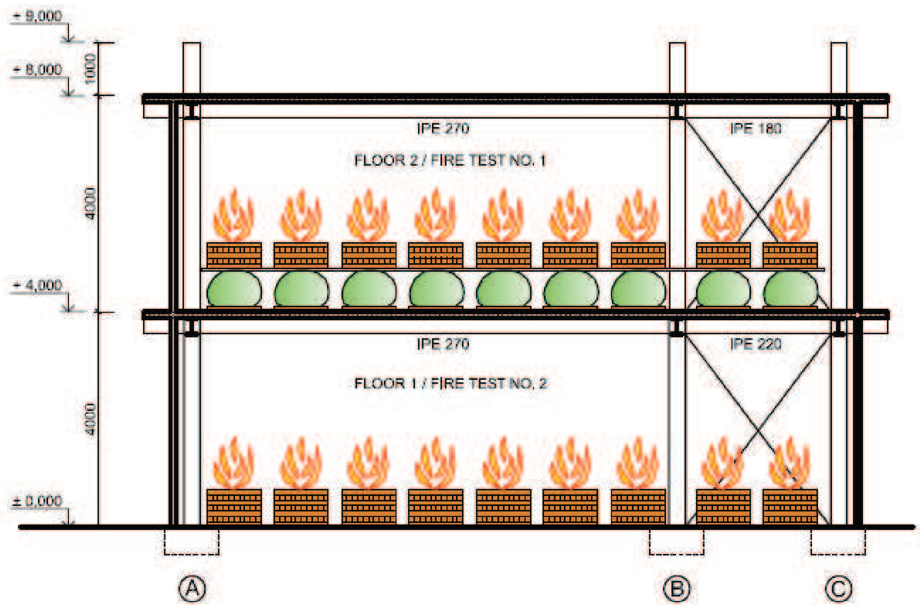


Figure 1. Test setup [5]

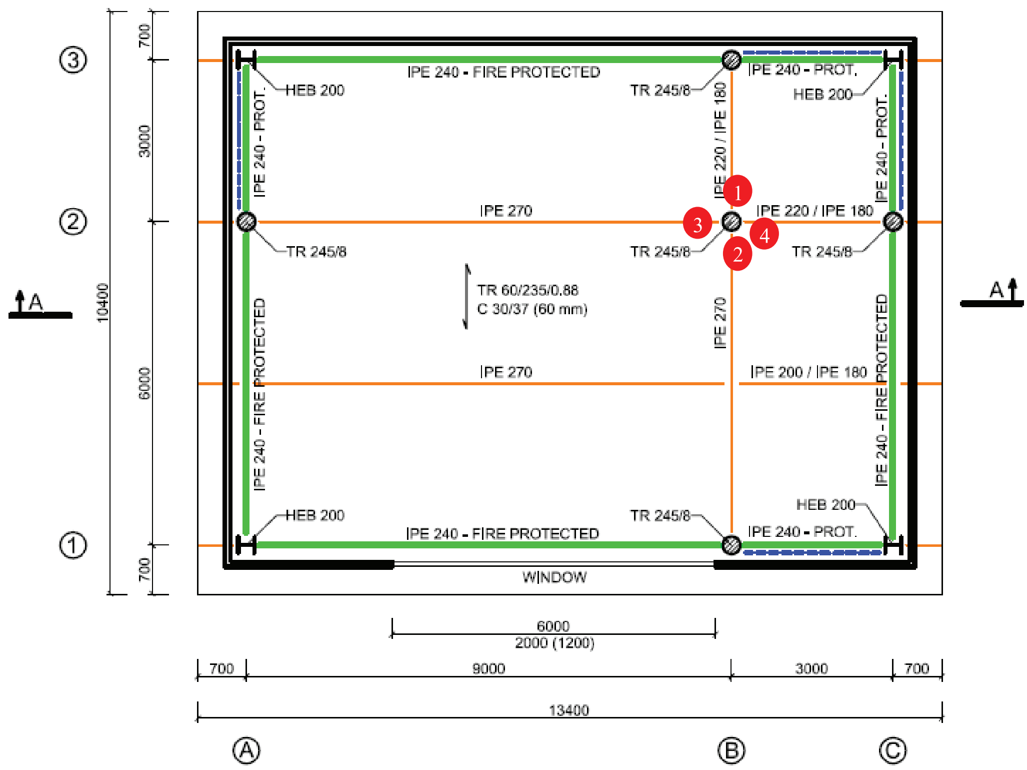


Figure 2. Floor plan of the test structure [5]

3.1 Model setup

Based on the design brief, the model was set up as shown in Figure 3. The 3D beam elements of *Vulcan* were used to model the beams, columns and bracing. The slabs were modelled by slab elements. A mesh sensitivity analysis was conducted, indicating that the beam elements should be between 1.0 m and 2.0 m in length and the slab elements should be approximately square. For simplification the flooring system and the beams of the upper floor were not modelled, but their self-weight (2.85kN/m^2) was applied on the column tops as concentrated loads. The applied load on the slab was 6.35kN/m^2, which was the overall actual (unfactored) load. The column base was assumed to be fixed. Since the frame was braced, the top ends of the upper-floor columns were restrained against lateral movement but free to deform axially, so thermal elongation was unrestrained. All the other elements had neither translational nor rotational restraints. While waiting for the actual temperature data, the first predictions were made by relating the beam, column and slab temperatures to a Eurocode 1: Part 1.2 [7] type of fire curve (Figure 4) which reaches 1000°C at 60 minutes and then starts to descend linearly. For the unprotected beams, the temperatures of their bottom flanges and webs were assumed to be 95% of the fire temperature, and the top flanges to be at 80% of the fire temperature. The temperatures of the bottom flanges and webs of the protected beams were assumed to be 50% of the gas temperature, and the top flanges to be at 45% of the fire temperature. The lower-floor columns and bracings, which were protected, were set as uniformly heated to 50% of the gas temperature. The temperature gradients through the slab depth were represented as a bilinear distribution, in which the slab lower surface was as hot as the fire; the temperature was 50% of the gas temperature 30% into the depth from the bottom, and the top surface reached only 15% of the fire temperature. The upper-floor elements were exempted from this assumption, as they were assumed to remain cool throughout the analysis. The recorded temperature data will be used as more accurate input data to the *Vulcan* model for further in-depth analyses.

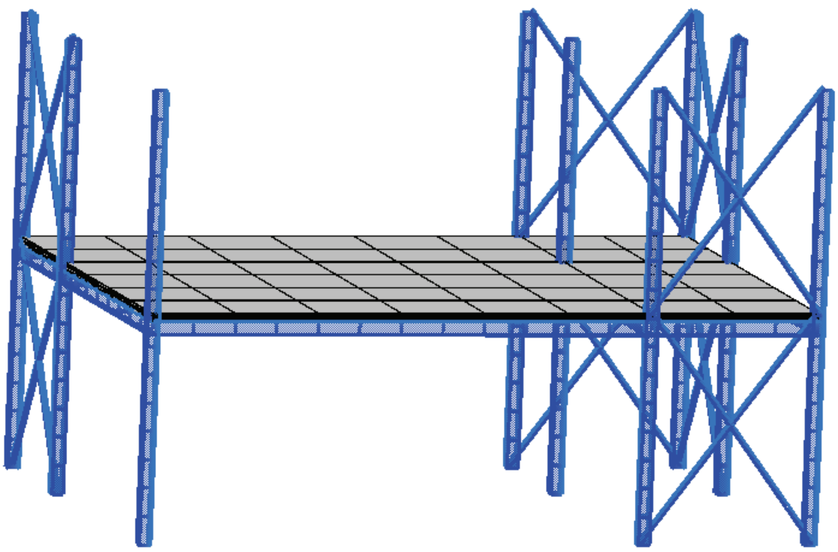


Figure 3. Vulcan model setup

For the diagonal bracing members, RHS 60mm x 60mm x 6mm of Grade S275 steel were assumed in the model. Due to the limitation on mesh shape in *Vulcan,* and the resulting complexity of modelling circular sections, the circular CFT columns were modelled as equivalent (in terms of cross-section area and second moment of area) square columns of 215mm width. Since the equivalent square section was found on the basis of both compressive strength and flexible stiffness, the effect of this simplification on the accuracy of the analysis is expected to be minimal. The connections were represented using rotational spring elements at this pre-test stage. As a part of the COMPFIRE project, a comprehensive component-based connection element is being established in *Vulcan*; the model will be upgraded using this connection element for the post-test simulation. Before the component-based connection element had been developed, the connections had to be simplified as rigid, pinned or semi-rigid by varying the rotational stiffnesses of the springs due to the limitations of *Vulcan*. The orthotropic nature of the slab was accounted for by using the *Vulcan* effective stiffness approach [8]. The full depth of the composite slab was modelled as a flat slab with different bending stiffnesses in the two orthogonal directions to account for the contribution of the ribs. The shear studs connecting the composite slabs and beams, each of which was assumed to have an ultimate shear strength of 350N/mm^2, were modelled using the shear-connector elements embodied in *Vulcan,* providing partial interaction between the slabs and beams.

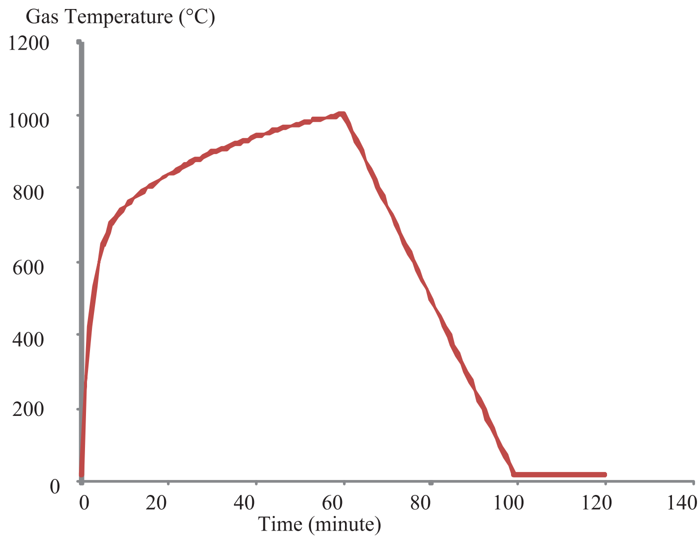


Figure 4. Fire curve

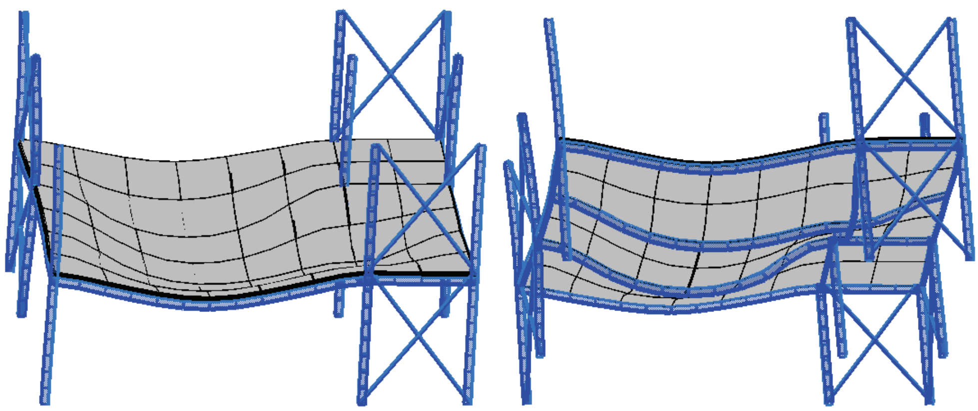
3.2. Results

The results of the *Vulcan* analyses are summarised in Table 1. Five models with different connection rigidities were analysed by varying the rotational stiffnesses of the spring elements, which were used to model the connections.

Tab. 1 Vulcan analysis results

Model No.	Connection Rigidity	Rotational Spring Stiffness (Nmm/rad)	Fire Resistance Period (minutes)	Max. Slab Displacement (mm)	Max. Connection Tying Force (kN)
1	Pinned	100	29	891	118
2	Semi-rigid	$5x10^5$	49	939	119
3	Semi-rigid	$1x10^9$	120	728	268
4	Semi-rigid	$1x10^{10}$	120	666	309
5	Rigid	$1x10^{12}$	120	642	354

The model with pinned connections failed at 29 minutes. Unsurprisingly, the fire resistance period was extended with increasing connection rigidity, and Models 3, 4 and 5 survived through the whole course of loading and heating without experiencing failure. The deformed shapes of the models are similar. One example is shown in Figure 5, in which the deflections are magnified three times. The centre of the slab panel A1-B3 experienced the highest vertical displacement. Figure 6 shows the development of the slab displacement at this point over time for each model, and the maximum displacement which occurred throughout the course of analysis is given in Table 1. The model with rigid connections deflected substantially less than the one with pinned connections, even though it was subject to much higher temperatures. On the other hand, Model 2 (with semi-rigid connections) experienced a larger deflection than Model 1 (with pinned joints), due to its extended fire resistance period compared to that of the latter.



(a) View from top (b) View of slab bottom

Figure 5. Deformed Vulcan model

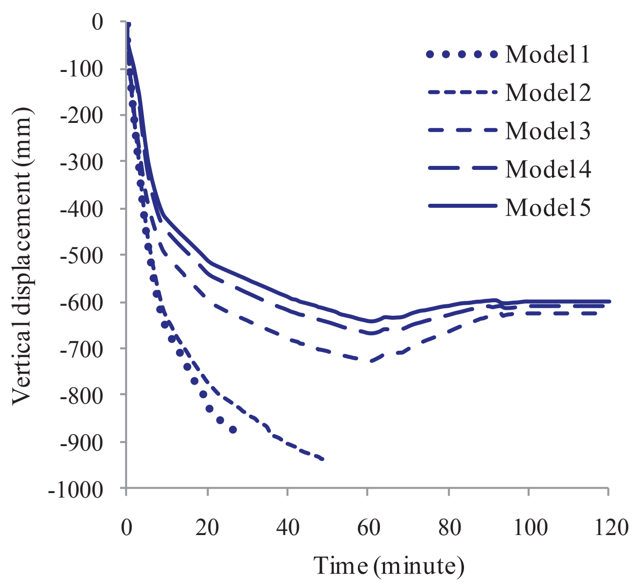


Figure 6. Maximum displacement of slab

Due to its higher structural continuity and exposure to higher temperatures, the maximum tying force generated in the connections of Model 5 (with rigid connections) was significantly larger than those of the other models, as given in Table 1 and shown in Figure 7. The forces in the connections to Column B2 of this model are plotted in Figure 8. The numbering of these connections is marked on the floor plan (Figure 2). The connections were initially in compression due to the restraint to thermal expansion, but as heating continues and the beams deform further, the compressive forces decrease. At 60 minutes, the heating reaches a maximum (see Figure 4), after which the beam deflection reduces (see Figure 6) and some tension in the beam is lost, thus slightly increasing the compressive force. As cooling continues, contraction of the permanently deformed beam causes further reduction of the compression and eventually the beam goes into tension. This phenomenon corresponds closely with the horizontal displacement of Column A2. It can be seen that, after an initial outward movement due to thermal expansion of the structure, this column moved inwards due to pull-in by the vertically-deflecting beams. This observation may prompt speculation about a possible cause of the failure of Models 1 and 2. Further in-depth analyses will be carried out, in the light of measured temperature and structural data.

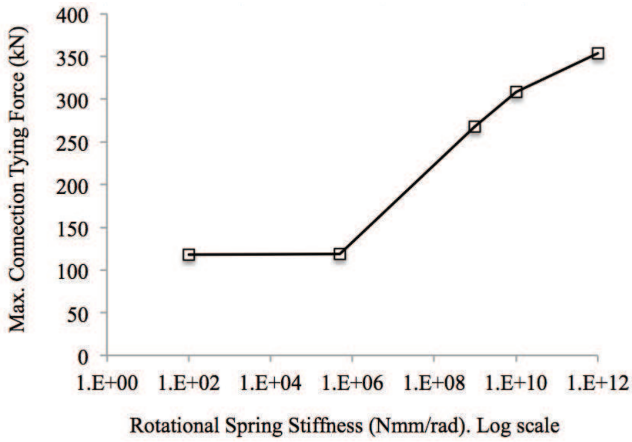


Figure 7. Maximum connection tying forces of the models with various connection rigidities.

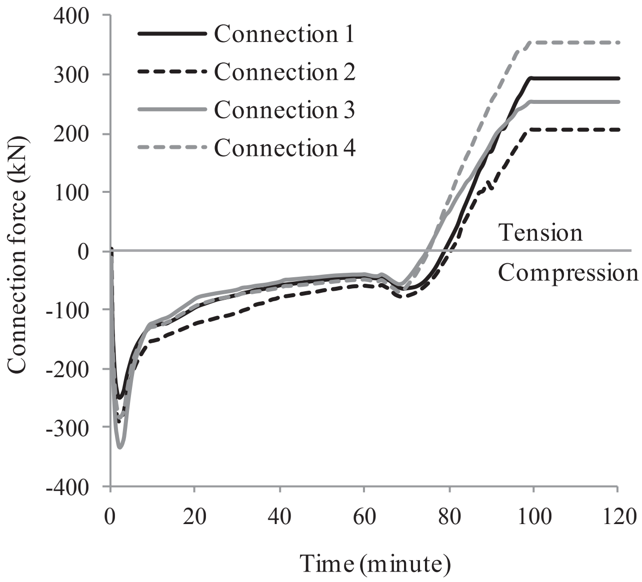


Figure 8. Connection forces of Model 5 (with rigid connections)

4. CONCLUSION

In this paper, an initial prediction has been made of the response to natural fire of the composite structure before it was tested in 2011 at Veselí using the finite element program *Vulcan*. While awaiting precise data (such as the temperature distributions) which was only confirmed when the tests were performed, conservative assumptions have been made at that pre-test prediction stage. Since the robustness in fire of the connections was of particular interest in these tests, five models of identical setup but different connection rigidities were analysed, and the following behaviour has been observed:

- The deformation shapes of the models are similar;
- Initially, the floor system expanded, which pushed the edge columns outwards and induced high compression in the connections due to restrained thermal expansion;
- This action reversed as the beam deflected further: the columns moved inwards due to pull-in by the deflecting beam, and the connection forces eventually reversed, becoming tensile;
- The effect of connection rigidity on the fire resistance and deformability of the structure is considerable; the fire resistance of the structure is enhanced by increasing the rotational stiffnesses of the connections;
- For the models with more rigid connections, the maximum tying forces in the connections are greatly increased, although the maximum slab displacements are not necessary smaller, since these models experience higher temperatures.

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