STRUCTURAL FIRE ENGINEERING ASSESSMENTS OF THE FRACOF AND MOKRSKO FIRE TESTS

An Engineering Prediction

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INTRODUCTION

The fire engineering of steel and composite frame buildings has become more and more standard practice in the UK in recent years. Simplified design methods allow structural engineers to omit fire protection from large numbers of composite beams. However, there are always buildings which fall outside the relatively tight boundaries of the simplified methods, and more advanced analysis approaches, normally implying the use of general or specialist finite element programs, are used. Although, these programs have been extensively validated during their development against available test data, the way in which a model is created and its results interpreted is extremely important. This was seen during the “Round Robin” CFD modelling of the Dalmarnock fire test [1]. Acknowledging that modelling of the dynamics of a fire is inherently less deterministic than that of the structural response of a building in fire, a similar lesson should be learned, as the effects of possible “modelling” mistakes could lead to catastrophic consequences.

As mentioned above, the FEM programs used to predict the structural response to fire have been validated against available test data. However, the bulk of the available test data comes from a series of just seven fire tests on a single building constructed in an old airship hanger in Bedfordshire, UK. The Cardington test building was designed as a typical composite frame building of the early 1990s, using standard UK building practice and details, which limits the available validation cases for the FEM programs to one particular type of construction. None of the tests led to the collapse of the building. The fact that the building techniques have developed further, and that finite element analyses of buildings in fire are conducted all over the world, means that programs are likely to be used outside the boundaries of the validations conducted. It is therefore even more important that parametric studies are carried out and that special care is given to the “modelling” assumptions and interpretation during the design process in order to give robust answers.

The year 2008, provided the opportunity for more diverse validation cases, with two full scale fire tests on parts of composite steel frame buildings. The first was the FRACOF fire test in Metz, France, in which a single slab panel with two unprotected secondary beams was tested under exposure to a 120-minute ISO834 Standard Fire. The second was the Mokrsko fire test, south of Prague in the Czech Republic, which exposed a purpose-built single-storey building to a natural fire. Buro Happold Ltd and the University of Sheffield used these opportunities to predict the structural behaviour prior to the tests, using the specialist finite element program *Vulcan*. During the assessments a number of parameters were varied within the normal range expected on site. For the FRACOF test two models were analysed before the test, with a more detailed follow-up in its aftermath. For the Mokrsko test the majority of the analyses were initiated before the test. The analyses were treated no differently from those for normal structural fire engineering projects, and it was expected to see conservative results. The results of the FRACOF assessment will be shown first, followed by the Mokrsko predictions.

1 THE FRACOF FIRE TEST

The FRACOF test was designed to demonstrate the benefits of incorporating tensile membrane action into the design of steel-framed composite floor systems in the European Community, and to assist in preparation of design guidance for its implementation. The test was therefore to investigate the performance of slab panels, as documented in the SCI document P-288 [2], and the effects of different construction details on their fire resistance.

1.1 Test description

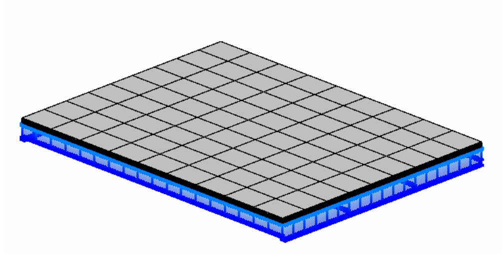
The test was set up as an 8.74m x 6.66m composite slab panel, representative of a corner compartment. It included four equally-spaced IPE 300 downstand secondary beams spanning in the longer direction, with IPE 400 primary beams. The floor arrangement was supported by HEB 260 steel columns, using simple connections. The slab was 155mm deep, on COFRAPLUS 60 decking, acting compositely with the steel beams. Beams and columns at the edge of the structure were wrapped in 50mm of Cerablanket protection (density = 128kg/m^3; specific heat capacity = 1130J/kgK; thermal conductivity = 0.06 – 0.2W/mK). Continuity across the two adjacent "internal" edges was simulated by welding the anti-crack mesh (7mm diameter bars at 150mm centres, placed 50mm below the top of the slab) to the flanges of horizontally-aligned HEB 200 sections before the concrete slab was cast. A gravity load of 3.87kN/m^2 was placed on the slab to simulate live loading at the fire limit state. The base of the structural assembly was exposed to the Standard Fire for 120min. Details of the test setup and results can be found in Reference [3].

1.2 Test predictions

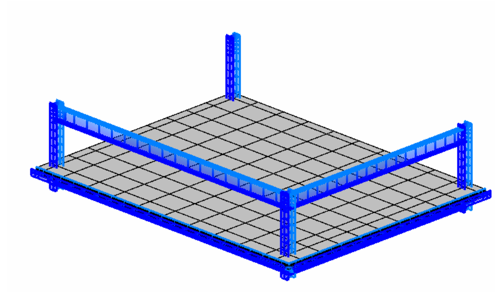
Five finite element test predictions are reported here; the first two were made before the test, with the subsequent three conducted afterwards, to correct differences in assumptions between the test design brief and the models. An overall slab thickness of 160mm had been specified in the brief, with no specific data on concrete strength. The applied load was given as 3.75kN/m^2, and it was assumed that the intended slab continuity would be achieved along the two adjacent "internal" edges. Based on the design brief and an assumed concrete cube strength of 40N/mm^2 the first predictions were made with protected beam and column temperatures following Eurocode 3: Part 1.2 [4] calculations, making a conservative assumption of Cerablanket thermal conductivity of 0.2W/mK. One-dimensional heat transfer was assumed for the concrete slab. The structural response predictions were made using *Vulcan* [5].

The first model considered the 8.74m x 6.66m slab as an isolated slab panel, supported vertically at its corners, with protected beams providing the necessary vertical support along the slab edges. The model used no axial restraints along its edges, but rotational restraints along two adjacent edges to simulate slab continuity across those boundaries. For conservatism, the 102mm thick continuous concrete layer above the decking troughs was modelled as a flat slab. The second *Vulcan* assessment used a full model of the test setup. It included the columns at the corners of the panel and the two horizontally-aligned HEB 200 sections along the "internal" adjacent edges for continuity. The orthotropic nature of the slab was accounted for by the using the *Vulcan* effective stiffness representation, developed by Huang *et al.* [6]. In this approach the full depth of the composite slab is modelled as a flat slab with different bending stiffnesses in the two orthogonal directions to account for the contribution of the ribs. The two models are shown in *Fig. 1*.

Test observations showed that the continuity condition was only practically achieved across the shorter edge. It was also observed that the protected beams and columns were not entirely within the furnace, and so did not attain appreciable temperatures or deformations.



(a) *Vulcan* slab panel



(b) Full *Vulcan* test setup

Fig.1. *Vulcan* models

This kept the slab panel boundaries supported vertically throughout the test. At 105mins however, fracture of a welded lap joint in the mesh caused an integrity failure of the central region of the slab. The *Vulcan* analyses are described in *Table 1*, and the central slab panel displacements predicted are plotted in *Figs. 2* and *3* together with the central vertical displacement from the test.

Table 1. Modified parameters in the *Vulcan* analyses

Parameter	V1	V2	V3	V4	V5
Concrete strength [N/mm^2]	40	40	37	37	37
Overall Slab thickness [mm]	160	160	155	155	155
Applied load [kN/m2]	3.75	3.75	3.87	3.87	3.87
Thermal conductivity [W/mK]	0.2	0.2	0.06	0.06	0.06
Protected beam temperature distribution	Uniform	Uniform	Uniform	Non-uniform	Non-uniform
Edge continuity condition	2 edges	2 edges	1 edge	1 edge	1 edge
Slab modelling approach	Thin continuous concrete	Effective stiffness	Effective stiffness	Effective stiffness	Average slab depth

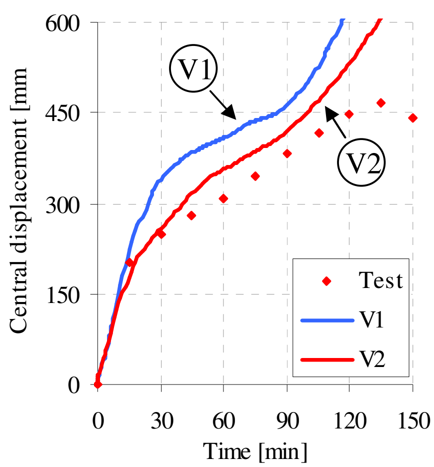


Fig. 2. *Vulcan* pre-test predictions

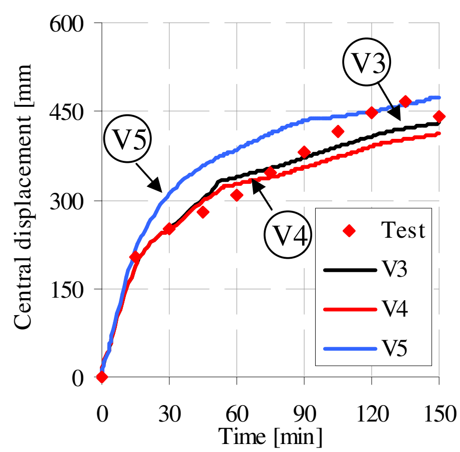


Fig. 3. *Vulcan* post-test predictions

It is observed that the initial predictions (*Fig. 2*) conservatively estimated the test deflection, although exact structural detail was not available. The subsequent analyses however showed better predictions (*Fig. 3*) using more realistic protected beam temperatures, non-uniform temperature distributions and the average slab depth approach. It is noticeable that the software's estimate of deflection worsened as the integrity failure point was approached.

2 THE MOKRSKO FIRE TEST

This fire test was conducted on the 18 September 2008 at Mokrsko, Czech Republic, by the Department of Steel and Timber Structures of the Czech Technical University of Prague.

The structure represented one floor of a steel and concrete composite office building consisting of four bays with a size of 9m x 6m each, and tested three different floor systems, namely "Angelina" composite beams developed by Arcelor-Mittal with elongated web openings, beams with corrugated webs made from thin steel plate, and precast hollow-core panels. The steel beams supported a composite slab with a total thickness of 120mm supported on CF46 metal decking. The slab was reinforced with a smooth mesh, providing a steel area of 196mm^2/m in each direction, situated 20mm from the top of the slab. The connections of the Angelina beams were by specially designed endplates which only connected the top flange and a small part of the web of each beam. The bases of the columns were constructed as pinned. The imposed load of 3.0kN/m^2 on the slab was generated by sand bags, and the self-weight of the floor system was 2.6kN/m^2. Timber cribs generated a total fire load of about 620MJ/m^2, and two 2.5m x 4m openings at the front provided ventilation to the fire. Steelwork fire protection was omitted from all Angelina beams, as well as the beams with corrugated webs. The rest of the steelwork was fire-protected using fire-board. This protection arrangement generated a 9m x 12m bay of unprotected Angelina beams, and a 9m x 6m bay of beams with corrugated webs, surrounded by protected beams. However, it left one edge column restrained in only one direction by fire-protected beams.

The fire burned a little cooler than expected, but after about 61 minutes three quarters of the structure collapsed; this is the only large-scale structural fire experiment which has generated a structural collapse. The corrugated web beams developed shear buckles near their ends, but their overall vertical deflections were relatively small; this can be explained by their greater depths and flange thicknesses compared with the Angelina beams. The Angelina beams showed severe Vierendeel bending across their first two openings, and after about 50 minutes the bottom flanges of some of the Angelina beams deformed laterally, folding the beams along their longitudinal axes. More details are given by Wald and Kallerová [7].

3.2 *Vulcan* modelling of the Mokrsko fire test

Before the test, the experiment was modelled using *Vulcan,* using only the fairly limited data available at the time. For simplification only the 3 bays with the composite slab were modelled, and the Angelina beams and the corrugated-web beams were represented using an effective web thickness approach which calculates a reduced web thickness based on the net cross section. This approach usually gives good overall results for beams with web openings but cannot adequately represent local effects around the openings.

Fig. 4. Fire test set-up

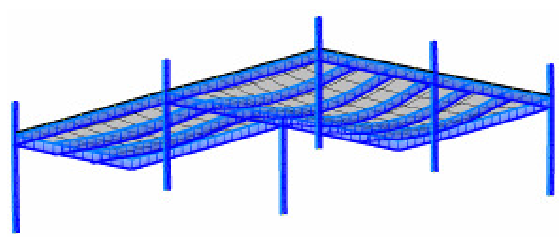


Fig. 5. Deformed Vulcan model

As with a normal structural fire engineering project a number of parameters were varied in order to test the robustness of the solution. Firstly, the fire was altered to produce a short-hot fire and a cooler-longer fire. The position of the reinforcement in the slab was then varied by ± 15mm to account for normal construction tolerances. The beam connections were modelled as rigid, which tends to be acceptable for normal composite connections designed to UK design rules in braced frames. Because of space restrictions here, only the results for the different fire curves are shown below.

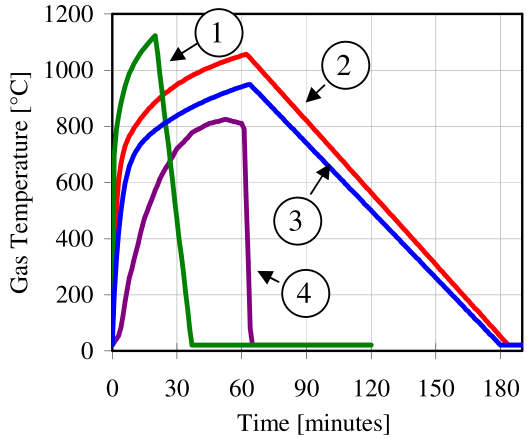


Fig. 6. Fire curves

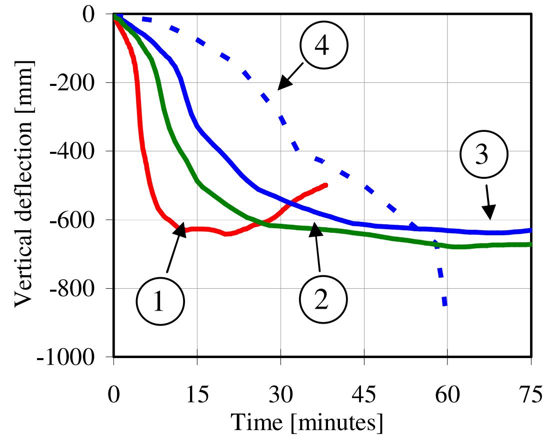


Fig. 7. Vulcan deflection prediction

It can be seen from *Fig. 6* that the real fire (4) in the experiment burned significantly cooler than the predicted fire (2). *Fig. 7* shows the resulting vertical deflections from the test and the three different design fires at the middle of the large bay of Angelina beams (V3).
The predictions show a much earlier increase in deflections than the experimental results. This is because the parametric fire curves represent post-flashover fires, and should therefore be moved by about 15minutes to give a realistic representation of the fire. This greatly improves the estimation of the fire test deflections. The models continued beyond the failure point of the test at about 61minutes, and do not show any indication of collapse, however the vertical deflections of the slab are in excess of span/15, which would normally result in an increase of reinforcement to limit the vertical deflections. Furthermore, all beams framing into columns would be protected in a robust design for fire.

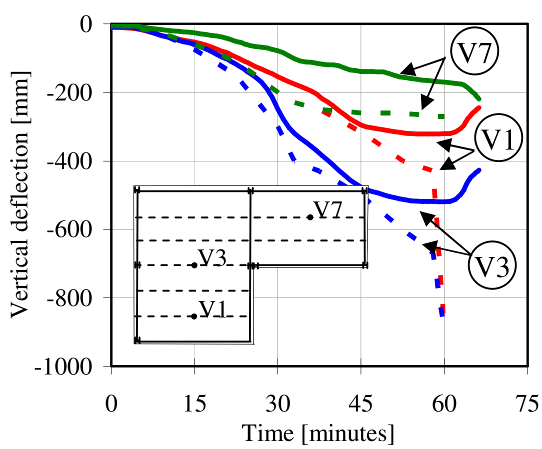


Fig. 8. Prediction using the average test temp.

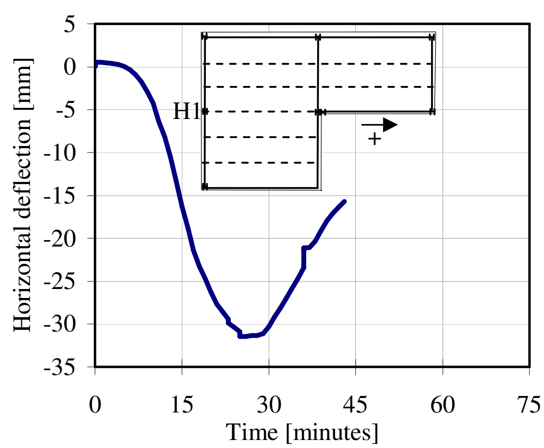


Fig. 9. Deflection of column head

After the test results were released, the actual temperature data was used as more accurate input data to the *Vulcan* model. *Fig. 8* shows that when the real temperature data is used the vertical deflections are represented relatively accurately up to about 44 minutes. The small vertical differences are due to the edge beam deflections, which are lower than those seen in the test, as well as to the use of average compartment gas temperatures to heat all elements. The difference between prediction and reality for the beams with corrugated webs (V7) can be explained by the observed shear buckling of the thin webs, which cannot be represented by the chosen way of modelling the beams.

Due to the very flexible beam connections, another set of analyses were conducted in which the connections were modelled as pinned. In these cases the *Vulcan* models predict failure at around 43 minutes using the experimental fire temperature data. From *Fig. 8* it can be seen that this is the point at which the *Vulcan* and test results diverge. *Fig. 9* shows the horizontal displacement at the top of the edge column connected to an unprotected Angelina beam. It can be seen that, after an initial outwards movement due to thermal expansion of the structure, the column moves inwards due to pull-in by the vertically-deflecting Angelina beams. This observation may prompt speculation about a possible cause of the test failure, but due to the lack of test data and further in-depth analyses at this time, this is not investigated further here.

4 CONCLUSION

In this paper, it is again confirmed that it is possible to make conservative overall predictions of the response of composite structures to fire using sophisticated finite element programs and that modelling can be accurate with accurate data. However, in both test cases it was not possible to predict the exact failure mode or time prior to the tests. With the accurate data given by the tests a fairly accurate representation of the structural behaviour can be made, and this implies that conservative assumptions will produce conservative predictions.

The integrity failure in the FRACOF test was undoubtedly related to the lap-welding of the mesh, but it will be necessary in future to develop programmable criteria for this local slab fracture. The unexpected collapse in the Mokrsko test is at present unexplained, but probably relates to construction details (pinned column bases, connections with limited tying capacity, columns connected to unprotected beams, poor connection between slabs and edge beams) which lack robustness. It is essential that robust construction details are developed and specified if fire protection is to be omitted from structural elements. If finite element analyses are used to justify the behaviour of non-standard forms of construction, which are most likely to lie outside the bounds of software validation, great care should be taken when modelling these problems, using detailed parametric studies and possibly even physical fire testing.

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