Shallow floor construction with deep composite deck: from fire tests to simple calculation rules

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This paper deals with an international research project into the fire behaviour of a particular, economically appealing, type of composite steel concrete beams: the shallow floor beams with composite slabs using deep deckings. The results of 3 full scale fire tests are briefly summarised. Based on these fire tests, advanced numerical models have been established enabling accurate simulation of the fire tests as well as a parametric study into relevant factors affecting the overall fire behaviour of this type of composite structures. The outcome of the parametric study is transformed into economic, yet easy to handle calculation rules for engineering practice.

Key words: **Shallow floor, deep composite deck, fire behaviour, finite element analysis, calculation rules.**

1 Introduction

Traditionally steel framed buildings consist of a steel frame work comprising columns, primary and secondary beams. If the steel frame is combined with a composite steel concrete slab, an economic load-bearing construction can be obtained, especially for high rise buildings. Recently, an alternative load-bearing construction has been developed, consisting of steel beams integrated in the floor slab. This paper is confined to the so-called SlimFlor design, which comprises a hot-rolled IS or RHS which is welded on a plate, on which a composite slab with deep steel decking is mounted. See Figure 1.

As can be seen from Figure 1, the shallow floor construction provides several advantages:
- a reduced floor depth;
- an almost flat underside of the floor which yields unrestricted layout for services;
- an inherent better fire resistance of the steel beams, due to thermal shielding by the surrounding concrete.

With respect to the latter point, it is noted that no specific rules are given for evaluating the fire resistance of these type of beams / slabs in Eurocode 4 part 1.2, although the general principles specified herein apply.

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Fire tests indicate that a fire resistance up to 60 minutes can be achieved without additional protective measures [1]. Longer periods of fire resistance can be achieved by, see Figure 2:

- protection of the bottom plate with insulation material.
- adding reinforcement.
- reduction of the degree of utilisation, or development of continuity at the supports.

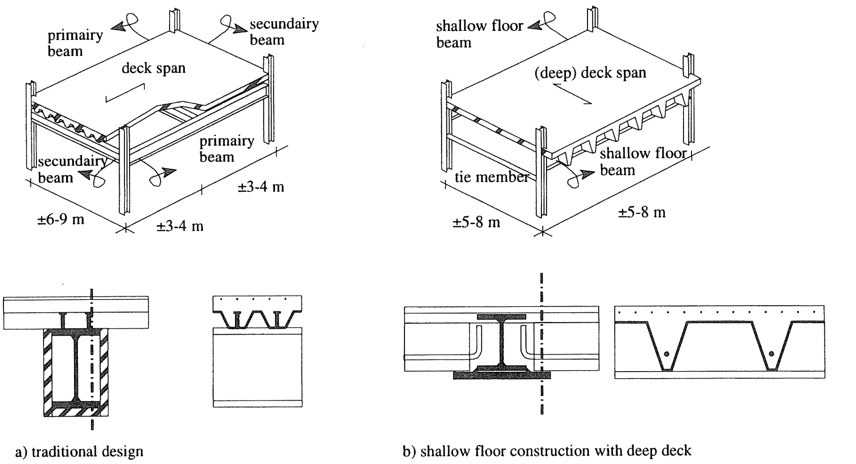


Fig. 1. *Steel framed buildings: traditional and shallow floor construction: inherent better fire behaviour steel beam reduction of total construction height.*

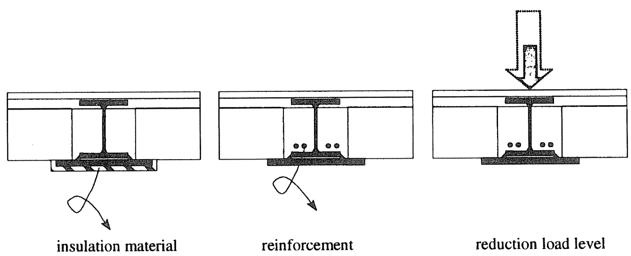


Fig. 2. *Methods for increasing the fire resistance.*

With a view to gain more insight into the behaviour of shallow floor systems, an international research project was performed by the Steel Construction Institute (UK) and TNO Building and Construction Research (NL). The final aim of the research project was to establish design rules. These rules were to be derived from parametric studies with numerical models, validated on the basis of tests. The total test programme contains tests at room temperature on separate members and system tests under fire conditions [1,2,8,9]. In this paper, the fire test programme, carried out at the Centre for Fire Research, is summarized. The numerical modelling of these fire tests is reviewed.

Finally, the process to achieve at simple calculation rules which allow easy assessment of the fire behaviour of shallow floor systems, is described.

2 Fire test programme

The fire test programme comprised the following fire tests:

- detail fire tests on three unloaded shallow floor beams to investigate the influence of the air gap between lower flange and welded plate (see Figure 3);
- two system fire tests to investigate redistribution of forces and flexural behaviour.

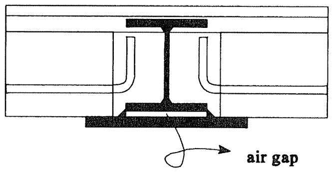


Fig. 3. Air gap between plate and I section.

2.1 *Fire tests on unloaded shallow floor beams*

During construction, i.e. welding of the plate to the steel section, a gap is likely to develop between lower flange and plate. Earlier tests have shown that significant temperature differences may occur (up to 300 °C), between the lower flange of the steel section and the plate [1,2,3], due to the inherent air gap. The instrumentation and measurements in these fire tests did not allow for an accurate determination of the effect of the different parameters.
Therefore, two test specimens have been prepared with deliberate air gaps of 2.2 and 4.1 mm.
A third test specimen was prepared for which efforts were made to minimise the air gap thickness.
All test specimens comprised of a HEM 180 welded to a plate of 400 × 15 mm. For a more detailed description of the test arrangement and test results, refer to [3].

From the test results, the following is concluded:

- significant temperature differences of maximum 200–270 °C may be expected (note that the thermal resistance of the air gap causes slightly increasing plate temperatures, but reduces significantly the lower flange temperatures);
- even in the test specimen in which the air gap thickness was as small as possible, significant temperature differences were found;
- the air gap thickness hardly influences results.

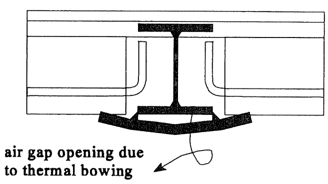


Fig. 4. Factors influencing the air gap thickness.

The explanation for the second conclusion is that due to thermal bowing the air gap opens. See Figure 4. So even for small air gaps, temperature differences are found.

2.2 *Systems fire tests*

Two fire tests were performed on complete floor systems. In these tests, the floor systems were exposed to ISO-fire conditions from below. The main purpose of the first test was to demonstrate the benefits of the composite action between slab and the integrated beam consisting of a hot rolled steel section with a steel plate welded to the lower flange. The main purpose of the second test was to obtain information on the behaviour of edge beams, consisting of a plate welded to a RHS section.

2.2.1 System fire test 1

The test was performed on a shallow floor system consisting of a continuous composite slab, using deep steel deckings supported on integrated steel beams [9]. For dimensions, refer to Figure 5 and Figure 6.

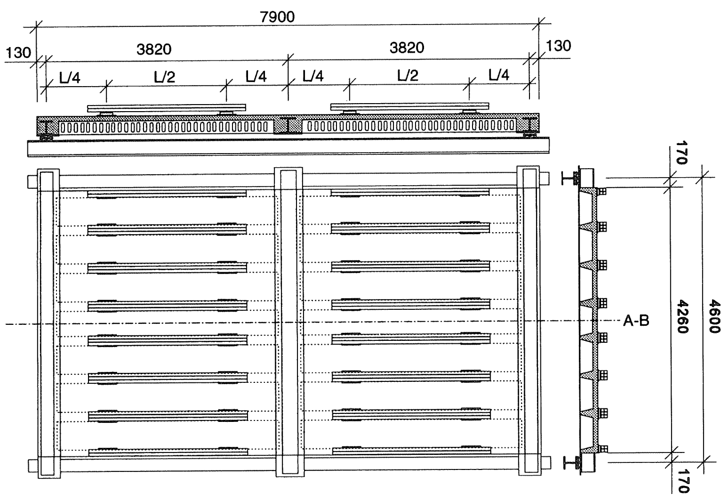


Fig. 5. Arrangement of the first structural fire test.

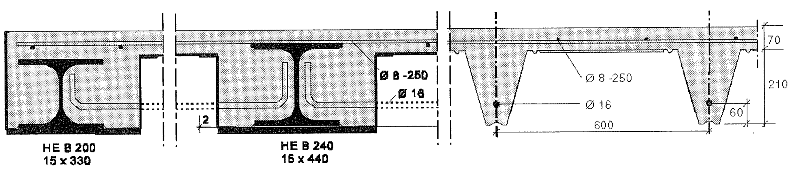


Fig. 6. Cross section of the beams and the slab in the first structural fire test.

A Precision Metal Forming LtD steel decking, type ComFlor 210/1.25 was used. The guaranteed yield strength of this type of decking amounts to 280 N/mm^2. The sheeting thickness was 1.0 mm including a zinc coating of 20 m at both sides. The concrete grade was C35. The rebars in the ribs and the reinforcement mesh on top of the slab were both hot rolled of grade FeB 500 HWL. The steel quality of all beams was S 235 (both I-sections and plates). Between the lower flange and the bottom plate of the centre beam a well defined gap of 2 mm was introduced, using iron chord parallel to the welds. These chords were melted into the welds. The minimal size of the fillet weld was 5 mm. No air gap was introduced between the bottom plate and the lower flange of the edge beam. A degree of utilization for the centre beam was approximately 0.5.

2.2.2 System fire test 2

The composite slab, which was almost identical to the slab of the first structural fire test, was supported on a bottom plate, welded on the edge beams, which were made of rectangular hollow sections (RHS). The test arrangement is presented in Figure 7.

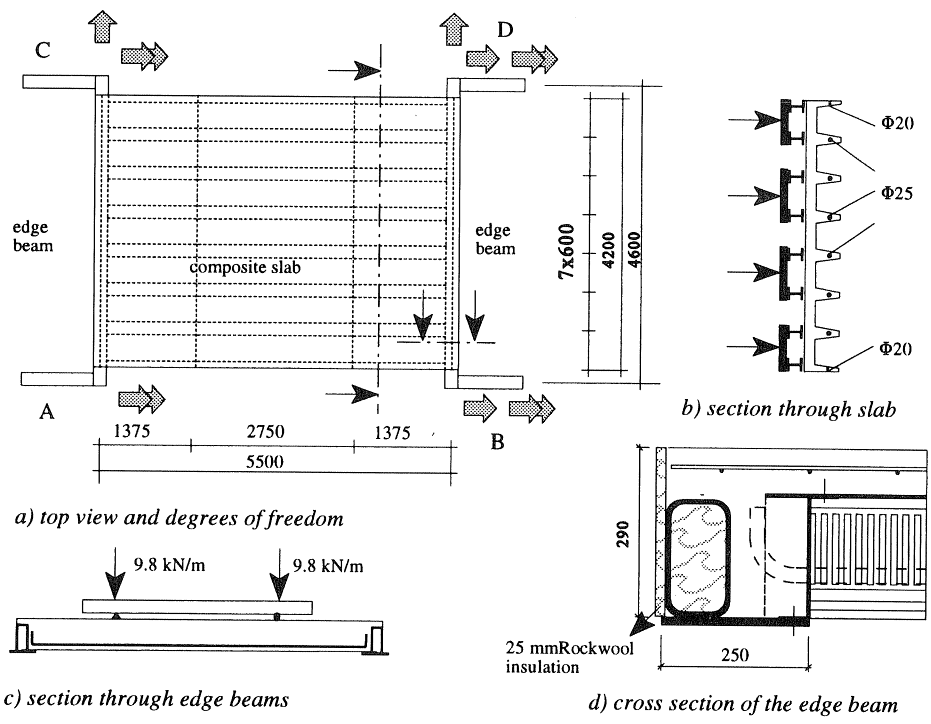


Fig. 7. Arrangement of test 2.

Normal weight concrete, class C35, was used. The thickness of the concrete topping on the upper flanges of the decking was 80 mm. In the six inner ribs of the composite slab, a hot rolled reinforcement bar $\phi 25$, quality FeB500 HWL, was applied. The axis distance with respect to the lower flanges of the steel decking was 75 mm. The bars were bent 90° at the supports in order to achieve proper anchorage. Furthermore, a cold worked mesh $\phi 6$–150, quality FeB500 HKN, was applied in the concrete plate, with a cover of 30 mm. The edge beams consisted of 200 × 100 × 10 rectangular hollow sections (RHS), quality S355. The bottom plate, quality S355, with a width of 250 mm and a thickness 15 mm, was welded to the edge beams; the minimum fillet weld dimension was 8 mm. No specific effort was made to ensure a gap between bottom plate and bottom flange of the RHS. The supports of the edge beams were designed such that no membrane forces could develop, and that rotation about the beam axes (torsion) was prevented. The degree of utilization of the edge beams was approximately 0.2.

3 Numerical modelling

The advanced calculation models were based on DIANA, a general purpose computer code using the Finite Element Method [5,6]. For the unloaded tests, only the thermal response was simulated. For the system tests, both the thermal and the structural response were simulated.

3.1 *Thermal response*

3.1.1 General assumptions

The temperatures in the centre beam and the slab were calculated separately by means of 2D models of the cross section. These models sufficed since the temperature measurements showed that the thermal gradient along the beam axis was negligible.

The following assumptions hold:

- All simulations were performed using quadrilateral linear elements (DIANA element type Q4HT).
- At the boundaries, heat transfer by means of convection and radiation was taken into account using one-dimensional boundary elements (DIANA element type B2HT).
- Convection coefficient at the exposed side: 25 W / m^{2}K (between the ribs of the slab 12 W / m^{2}K); at the unexposed side 8 W / m^{2}K. Radiation: (resulting) emissivity of concrete 0.78; steel beam 0.6; steel sheets 0.09 (≤ 250 °C)–0.4 (≥ 800 °C) accounting for the melting of the zinc coating. The bottom plates of the unloaded test specimens were of blanket steel, with a resulting emission factor that developed between 0.3 (20 °C)–1.0 (≥ 800 °C).
- In voids, radiative and convective heat exchange between the surfaces at the void envelope was accounted for by means of a subroutine suitable for convex voids [7].
- The thermal properties of steel and concrete were taken from Eurocode 4 [4].
- Evaporation of free moisture was modelled by a modified heat capacity of concrete.
- Debonding of the steel sheet is taken into account after reaching a temperature of 100 °C, (i.e. the void is assumed to be initiated by the evaporation of moistures).

3.1.2 Beams

In view of symmetry, only one half of the cross section needed to be modelled, see Figure 8.
The air gap between the bottom plate and the lower flange was modelled, taking into account the conduction of air and the radiation between the envelope of steel surfaces.
Perfect contact is assumed between the bottom plates and the hollow section edge beams of the second system test. In Figure 9, the adopted mesh is shown. Note the crack between the edge beams and ribs of the slab. This separation was assumed to exist already at the start of the simulation.

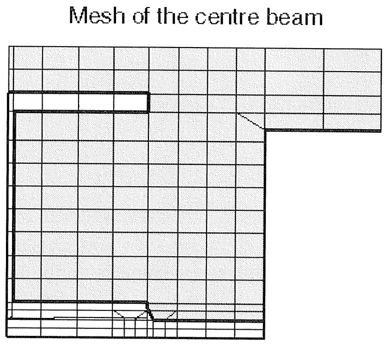
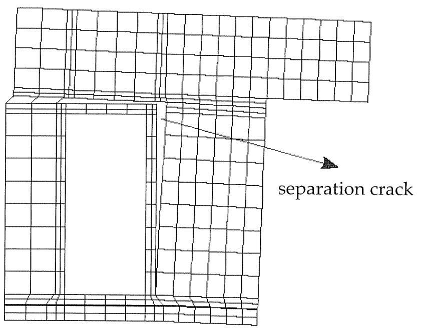


Fig. 8. Mesh of the I-shaped beams used to simulate the thermal response.



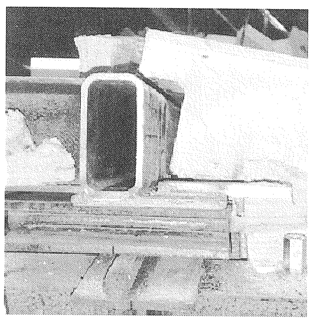


Fig. 9. Separating crack in edge beam of second system fire test and adopted mesh for the simulation of the termal response of the edge beams.

By way of illustration, Figure 10 shows the calculated and measured temperatures as a function of the time of the rectangular hollow edge beams of the system test. The calculation is in good agreement with the measured temperatures. Hence, the calculated temperatures are a good basis for the structural modelling.

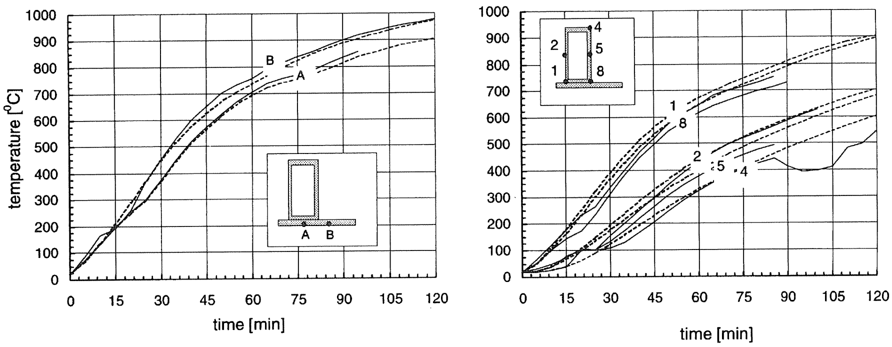


Fig. 10. Comparison between the measured and calculated temperatures in the rectangular hollow edge beams of the second system test.

3.1.3 Composite slab

The slab model made use of the two lines of symmetry of the slab, i.e. in the heart of the rib and in the middle between two ribs, See Figure 11. The web and the upper flange of the steel sheet were exposed to less severe heating, since the heating depends to a large extent on the radiative heat exchange. These effects are incorporated by modelling a void closed by a perfectly dark surface with a temperature development according to ISO 834 and a perfectly reflecting surface at the line of symmetry between two ribs.

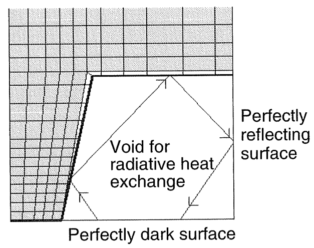


Fig. 11. Mesh of the composite slab used to simulate the thermal response (notice the debonded steel decking).

The sudden increase of the temperature difference between the steel sheet (upper flange and web) and the concrete cover which was measured between 15–20 minutes at approx. 100 °C, indicates that the contact between the steel sheet and the cover got lost, presumably due to evaporation. After the test a gap between the sheet and the concrete was observed. Analyses of the structural behaviour of the floor system showed that a proper assessment of the temperature of the steel sheets is of vital importance for the thermal gradient of the slab and thus for the global load redistribution. To simulate the thermal response of the slab, a void is modelled between the steel sheet and the concrete cover, including radiative heat transfer and conduction. Up to 100 °C no thermal resistance of this air layer is assumed, by setting the conduction of air at infinity.

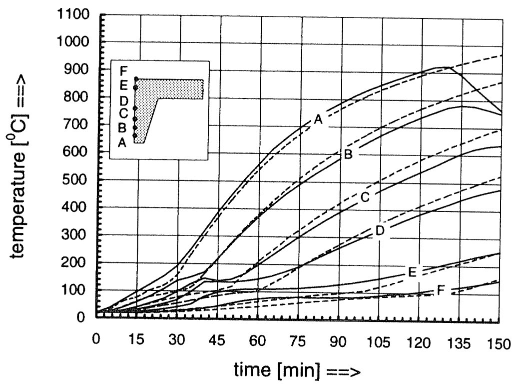


Fig. 12. Measured (solid) vs predicted (dashed) thermal response of the composite slab.

In Figure 12 the measured temperatures in the rib are compared with the calculation results. The agreement is good, which is particularly important for the temperatures of the rib rebar (measuring point D).

3.2 *Structural response*

3.2.1 General assumptions

The following assumptions hold:

- The beams in system test 1 and the composite slab in both system tests were modelled with Mindlin beam elements (DIANA element type CL18B). The edge beams in test 2 were modelled with shell elements (DIANA element type CQ40S).
- The temperature dependent mechanical material properties of steel and concrete were taken from Eurocode 4 [4], accounting for actual mechanical material properties at room temperature.
- Yielding of concrete in compression was not modelled in order to avoid numerical instabilities; earlier work by Jaarsveld has shown that this simplification does not influence the results significantly [12].
- The simulations were performed using a secant (BFGS) iteration method; a standard Newton Raphson scheme appeared to fail easily, due to complex concrete cracking phenomena.
- The calculations were based on the temperature simulations as presented in the previous paragraph.
- Full composite action was assumed between concrete and steel in the beams of test 1 and in the composite slab; in test 2, for the edge beams, composite action was completely ignored.
- Only physical non-linearity is taken into account; although the deflections are large, it is assumed that geometrical non-linearity would not improve the results because, due to special provision in the test arrangement, no significant axial forces will occur.

3.2.2 System fire test 1

In view of symmetry only one half of one slab span needed to be modelled. See Figure 13. Note that no plate action was taken into account because the slab was modelled with separate beam elements for each ribs without longitudinal joints. The composite action between the slab and the beam was taken into account by introducing an effective width of b_{eff} equal to $L_{beam}/4$ for the beam, see Figure 13.

In Figure 14, the measured support reaction is compared with the simulation results. The agreement is good. In the first hour, the simulation is in good agreement with the measurements.

The test results – supported by the numerical simulation – show that the bare steel construction consisting of a two-span continuous compo-site slab on integrated steel beams reached a fire resistance of more than 90 minutes in a full scale fire test. For a more extensive comparison of experimental and numerical results refer to [3].

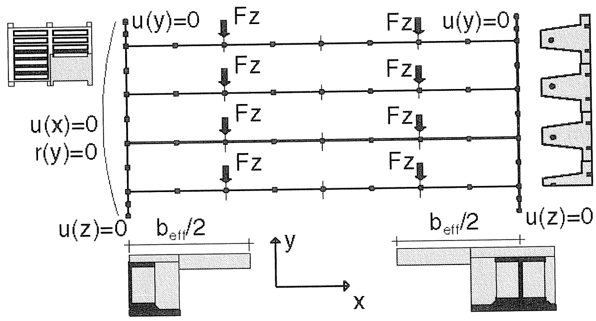


Fig. 13. Overview of the mesh used for the structural response (z-direction is perpendicular to drawing plane); system test 1.

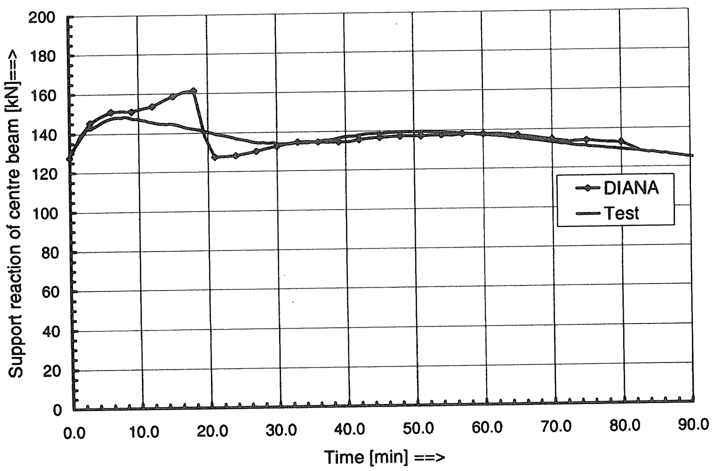


Fig. 14. Comparison measured and simulated support reaction system fire test 1.

3.2.3 System fire test 2

In order to calculate the mechanical response of the system, only one quarter of the specimen needed to be modelled (symmetry, see Figure 15). The ribs of the composite slab are assumed to act separately: each rib is modelled with Mindlin beam/column CL18B elements. The hollow section edge beam is modelled with curved shell CQ40S elements.

In Figures 16a,b the calculated mid span deflections of the edge beams and the composite slabs results are compared with the experimental results. After approximately 55 minutes, the edge beams made contact with a safety support stud, which was provided in furnace, in order to prevent damage. Hence, beyond 55 minutes, no test results are given in Figure 16a.

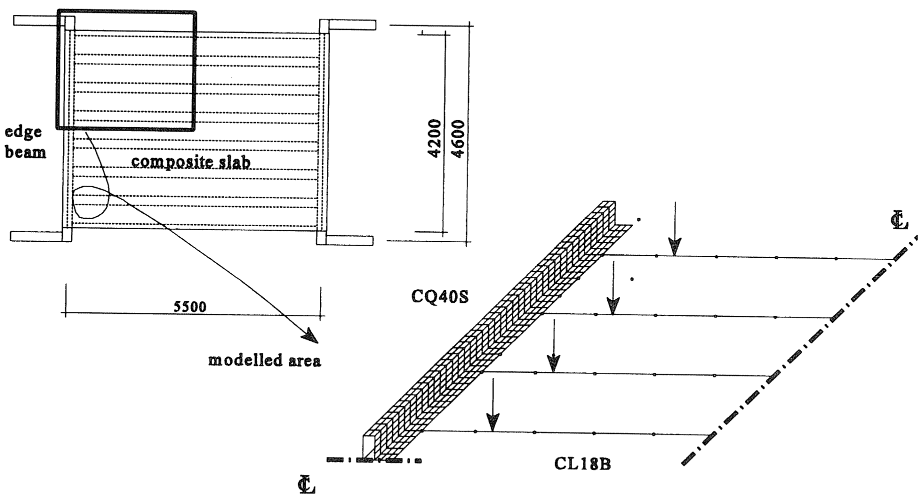
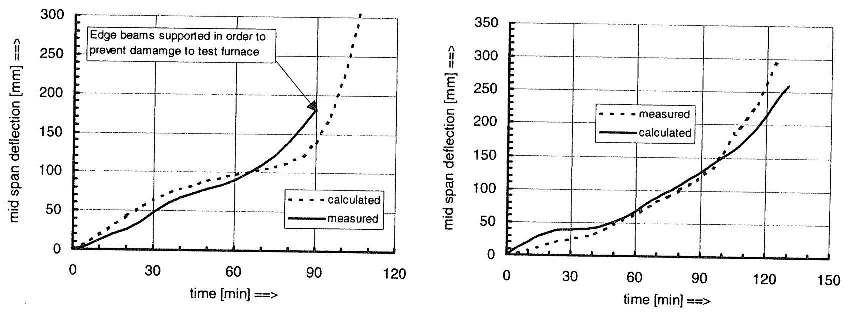


Fig. 15. Overview of modelled area and finite element mesh for system list 2.



(a) Midsapn deflections in the edge beams *(b) Midspan deflection of the composite slab*

Fig. 16. Measured (solid) vs calculated (dashed) deflections: system fire test 2.

4 Parametric study and simple calculation rules

With aid of the validated FEM models, simplified calculation rules for the thermal response of slim floor beams and composite slabs with deep steel decking are developed by means of a parameter study. For the integrated beams, the difference between normal and light weight concrete is assumed to be very small and is, therefore, not investigated. For composite slabs, separate calculations were performed.

In the parameter study into the fire behaviour of integrated beams, the dimensions of the section and the plate were systematically varied in order to find a relationship with the temperatures in the bottom plate and the section. For the cross sections of the Slim Flor beams without an air gap, and for the RHS edge beams, no thermal resistance is assumed between the plate and the lower flange. However, still a temperature difference between the plate and the bottom flange is found.
By way of illustration, the accuracy of the simple calculation rules is demonstrated in Figure 17. In this figure, the temperatures of the lower and upper flanges of unprotected integrated RHS edge beams for fire exposure times in the range of 30 to 120 minutes assessed with the simple calculation rule are compared with the outcome of the advanced numerical model for the thermal response. It can be seen that the agreement is very good: the difference between both calculations remain within 5 %.

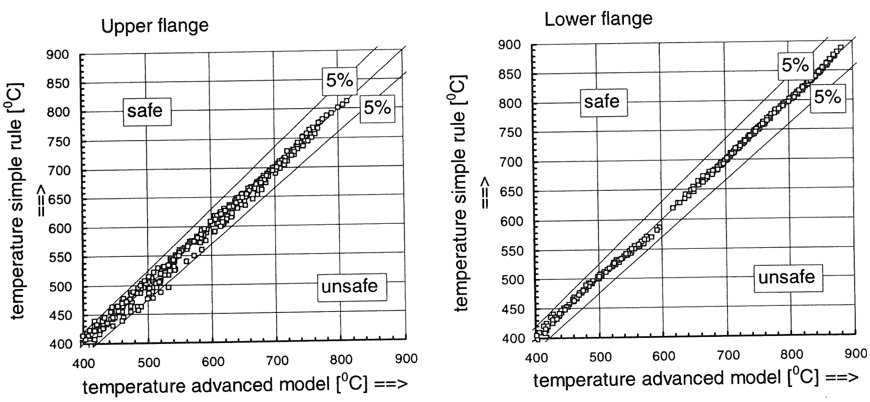


Fig. 17. Temperatures calculated with advanced numerical model versus temperatures obtained with the proposed simple calculation rule for RHS shallow floor beams.

The plastic sagging moment capacity of the integrated beams and the composite slab can easily be calculated on the basis of a plastic analysis of the cross section (stress-block method), taking account of the temperature dependent material properties as specified in ENV-1994-1-2. The strength of concrete in compression may be assumed to be unaffected, provided that the insulation criterion is met. Since tests and calculations have shown that the temperature of the steel deck after 60 minutes of fire resistance are over 900 °C, the contribution of the steel decking to the load-bearing capacity may be neglected. For continuous spans, sufficient rotation capacity above the intermediate supports should be provided. For a detailed review of the simple calculation rules, refer to [2].

5 Conclusions and recommendations

Based on the relationships for the relevant temperature dependent material properties as specified in Eurocode 4, the thermal and mechanical response of the tested fire exposed Slim Flor systems, comprising of normal weight composite slabs with deep steel decking, can satisfactorily be predicted, using the general purpose finite element programme DIANA.

In order to predict the temperatures in the steel decking of the composite slab, it is necessary to assume a thermal resistance between sheet and concrete due to loss of contact. This has been done successfully by means of a pilot version of DIANA, in which heat transfer by means of radiation and convection in voids was added to the potential flow module. The assumption that this loss of contact is initiated by the evaporation of moisture and thus occurs at a temperature of 100 °C, gives a reasonable agreement, experimental and theoretical outcomes.

With the validated numerical models for the thermal response, a parameter study was performed, from which simple calculation rules were deduced which allow for fast and economic assessment of the fire resistance of Slim Flor systems.

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