Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte
Band: 999 (1997)

Artikel: Compression membrane action in composite slabs
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DOI: https://doi.org/10.5169/seals-999

Compression Membrane Action in Composite Slabs

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Summary

This paper discusses the results from a two year research project which studied the effects of Compressive Membrane Action (CMA) on composite metal decking/concrete slabs before and after fire. The paper concentrates on work carried out on full scale slabs in the BRE test building at Cardington, but compares these results with those obtained from slab strips tested in the laboratory. It was found that fire damaged slabs exhibited far greater strengths than previously supposed, some held loads higher than those predicted by yield line analysis.

1. Introduction

Compressive Membrane Action is the two way arching effect which occurs when a laterally restrained slab is loaded (1). The load is resisted by a compressive force which extends through the slab from the load to the supports (see Fig 1). The greater the element depth/length ratio is, the greater is the amount of arching which occurs. CMA is already used to justify a reduction in the amount of reinforcing steel used in certain beam and slab bridge decks (2), but has not yet been applied in practice to other structures (3). The purpose of this research was to find out whether CMA contributes to the strength of composite metal decking/concrete slabs in buildings, and whether it helps to sustain load in a composite slab which has been badly damaged by fire (4).

Thus, a series of full scale in-situ and laboratory tests was devised to investigate the effects of the following parameters:

a) slab boundary conditions - interior, edge and corner composite slabs were tested and
b) fire damage - composite slabs were tested before and after fire loading.

2. Tests at BRE Cardington

The full scale in-situ tests were carried out in the Building Research Establishment's 'Large Building Test Facility' (LBTF) at Cardington, UK. The building is eight storeys high and replicates a typical steel framed, composite slab office block with a design dead load of

3.65kN/m^2 and design imposed load of 3.5kN/m^2. It has been built purely for research purposes, in particular to assess the effect of fire on a 'real' building.

The tests were carried out in two phases, the first on undamaged slabs (pre-fire tests) and the second on fire damaged slabs (post-fire tests) at the locations indicated in Fig 2. Each set of slabs underwent two tests, the first being a proof load test and the second an ultimate load test. Loads were applied along the quarterpoints of the slab in order to simulate the bending moments due to a uniformly distributed load. The load was applied hydraulically using ten 30 tonne jacks (Fig 3), except for the service load on the fire damaged slabs which was applied incrementally with 1.1 tonne sandbags. This was mainly because the panels were so distorted by the fire that it would have been difficult to line up holes for the Macalloy bars over two floors. In addition, it was known that the slabs had sustained a superimposed load of about 2.4kN/m^2 during the fire test and it was decided not to load the slab more than this in case the beam connections failed.

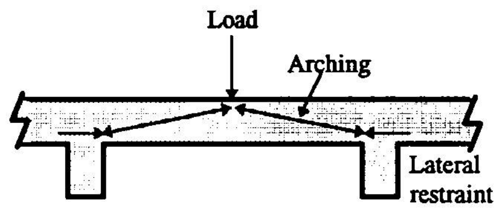

Fig.1 Arching action

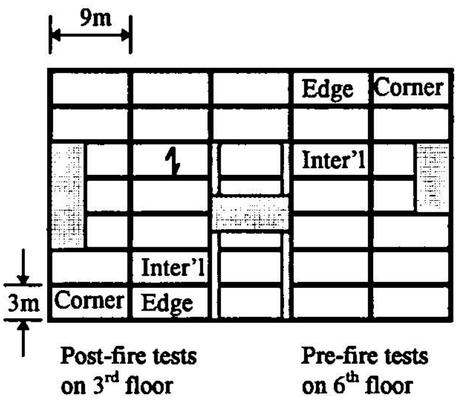

Fig.2 Cardington test panels

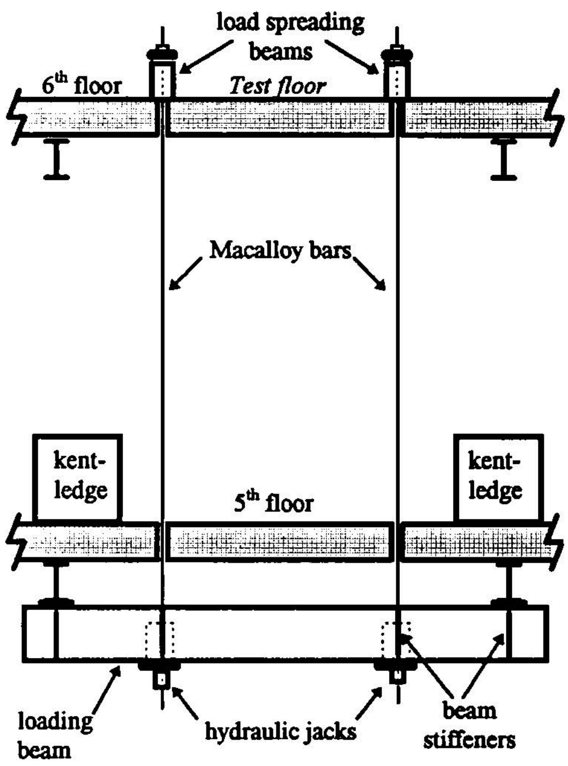

Fig.3 Hydraulic floor loading system

2.1 Proof load tests

2.1.1 Pre-fire proof load tests

The slabs were first loaded in 1.2kN/m^2 increments up to an applied test load of 5.6kN/m^2 (1.25 Dead load + 1.25 Imposed load - slab self-weight - weight of test apparatus). Deflections were measured at each increment of load with a staff and level. This test was carried out twice and then the load was allowed to remain on the slab for 24 hours. The deflections were taken before and after to ascertain whether any significant creep had occurred. The slabs deflected less than

1/400 of the 3m span under the imposed load, and the 24 hour tests showed that no creep occurred.

2.1.2 Post-fire proof load tests

Before these tests began, the slabs were subjected to a fire load of up to 763°C. This procedure is described in more detail in the BRE's newsletter, LBTF News, Issue 13, Summer 1996. For this test it was decided to only load the slabs once as the floor was fairly badly damaged, and moving the sandbags on and off the floor was causing the cracks along the edge of the slab to open up further. The load was taken up to a superimposed UDL of 2.4kN/m^2. Although this load was less than half the load for the pre-fire tests, the deflections were fairly similar, indicating that the slab was not maintaining load as well as before the fire. However this was to be expected, as the steel frame had badly deformed, making the whole structure much less rigid.

Slab location	Pre-fire tests: Deflection under 5.6kN/m^2 imposed load (mm)	Post-fire tests: Deflection under 2.4kN/m^2 imposed load (mm)
Internal	7	6
Edge	4.5	4
Corner	Test not carried out	5

Table 1 Comparison of slab central deflections before and after fire

2.2 Ultimate load tests

The test rig set up was slightly different from the proof load tests (Fig 3), because only the slab was to be loaded to failure. As it was imperative that the steel frame did not fail, the test rig was moved up one floor so that it was immediately under the slab being tested. This created a self-straining system whereby the load on the slab was resisted by the reactions of the loading rig against the steel beams.

2.2.1 Pre-fire ultimate load tests

The slabs were loaded in 1.2kN/m^2 intervals, until they approached failure when the loading increments were reduced to 0.6kN/m^2. At a load of about 15kN/m^2, the decking began to debond noisily from the concrete. It is likely that this was the point at which the concrete began to crack as well, but this was impossible to see with the decking in place. At this point, the stiffness of the slab reduced by about 60%. At about 2/3 of the ultimate load level, yielding of the decking began to occur and the rate of deflection progressively increased until failure.

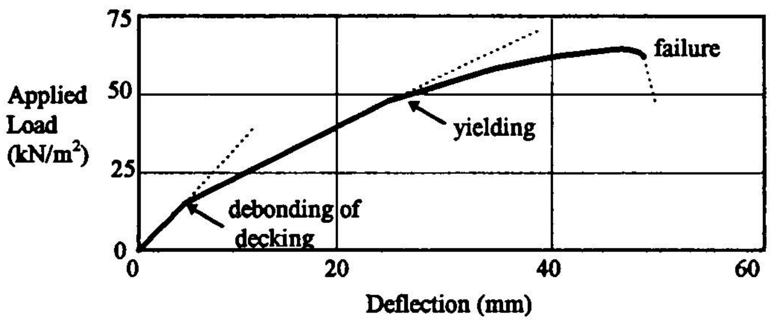

Fig. 4 Pre-Fire Load/Deflection Results for Internal Panel

In all cases, failure consisted of a sudden, shear failure along one of the edges parallel to the spanning direction of the slab. The failure loads are shown in Table 2. It can be seen that the strengths of the panels (except the post-fire internal panel) were considerably greater than the predicted yield line loads, particularly for the pre-fire internal panel.

In general, the failure load was related to the depth of the slab, which varied across the floor. However, the internal panel sustained a significantly higher load than the edge and corner slabs, even though it was the least deep. This implies that the internal slab was restrained to a greater extent by the surrounding slabs and beams indicating that there was a greater contribution of CMA to the load carrying capacity of the internal slab.

2.2.2 Post-fire ultimate load tests

The fire load caused the slabs to crack along the beam lines. The slabs had deflected with the steel beams which had deformed greatly due to the fire load. In these tests the decking was not heard to debond - presumably because this had already occurred under the fire load. As failure was approached, the load was carefully controlled so that the slabs did not fail catastrophically.

A summary of the failure loads is given in Table 2. The edge and corner slabs were not as badly damaged as the internal panel in the fire and their failure loads were correspondingly higher. The internal panel had deflected by nearly half a metre in places and so was much less rigid than in the pre-fire tests. It is interesting to note that, even after the fire, all the panels withstood at least three times the required design ultimate load of 10.7kN/m^2.

Slab location	Cardington failure load - P_{CAR} (kN/m^2)	Yield line load - P_{YL} (kN/m^2)	P_{CAR}/P_{YL}
Pre-fire			
Internal	69.9	37.4	1.87
Edge	61.4	39.2	1.57
Corner	62.4	41.9	1.49
Post-fire			
Internal	38.0 (54%)*	38.9	0.98
Edge	46.4 (75%)*	38.5	1.20
Corner	47.4 (76%)*	37.0	1.28

** % of pre-fire failure load*

Table 2 Cardington Test Results and Theoretical Failure Loads

3. Laboratory Tests at Queen's University, Belfast

Four tests were carried out to assess the effect of fire on a slab. Two of the slabs were simply supported and two were laterally restrained using a rig that had been built for the purpose. The specimens were designed to be similar to the slabs at Cardington, so were made 3000 mm long and 130 mm deep. Comflor CF70 metal decking was used, as at Cardington, and a typical decking profile is shown in Fig 5. Debonding of the decking due to fire damage was simulated by greasing the decking with a release agent before casting the slab. The results of the tests are shown in Table 3.

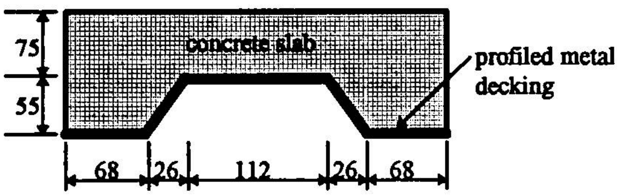

Fig.5 Cross-section of composite slab strip

The slabs were loaded incrementally to failure using a Dartec electro-hydraulic actuator, and readings of load and central deflection were taken at each increase in load. For the pre-fire tests the decking could be heard debonding from the slab at a load of about 14.4kN/m^2 which is similar to the level of load at which this occurred in Cardington. The post-fire simulations showed that the slab still had considerable strength, even though the decking was not bonded to the slab. This agreed with the full panel results from Cardington.

Slab type	Failure load (kN/m^2)
Pre-fire tests (with decking)	
Simply supported	34.3
Restrained	65.9
Post-fire tests (debonded decking)	
Simply supported	27.9
Restrained	34.5

Table 3 Laboratory Test Results

4. Comparison of Results

The Cardington test results are compared with the laboratory test results in Table 4. The failure loads for the Cardington slabs are first compared with the failure loads for the simply supported laboratory strips. The ratios of $P_{CAR}/P_{LAB(SS)}$ show that the strength of real panels tested in-situ was considerably greater than the conventional simply supported design strength. This strength enhancement is mainly attributable to the presence of rotational and lateral restraint at the slab boundaries which gives rise to boundary moments and Compressive Membrane Action. As the internal panel had the greatest degree of restraint it exhibited the greatest strength enhancement (except in the post-fire test where the internal panel was the most severely damaged by the fire).

The ratios of $P_{CAR}/P_{LAB(RES)}$ for the pre-fire tests show that the high degree of restraint used in the laboratory was close to that of the internal panel at Cardington but was greater than that for the edge and corner panels. The post-fire test results again reflect the more severe damage to the Cardington internal panel than the edge and corner panels and also indicate that the edge and corner panels had greater residual strength than the laboratory results suggested.

All of the slabs exhibited strengths far greater than the ultimate design load of 10.7kN/m^2, which shows that even after a fire, although the slab would be unserviceable, it may be possible to rely on the residual strength of the slab for safety purposes.

Slab type	$P_{CAR}/P_{LAB(SS)}$	$P_{CAR}/P_{LAB(RES)}$
Pre-fire		
Internal	2.04	1.06
Edge	1.79	0.93
Corner	1.82	0.95
Post-fire		
Internal	1.36	1.17
Edge	1.66	1.43
Corner	1.70	1.46

Table 4 Comparison of Cardington and Laboratory Test Results

5. Conclusions

- The strengths of the composite metal decking/concrete slab panels in the BRE Cardington test building were found to be significantly greater than the ultimate capacities predicted by yield line theory. This strength enhancement is mainly attributable to the effect of boundary conditions which induce boundary moments and Compressive Membrane Action.

- The load capacity of composite slabs that have been subjected to an intense fire is primarily reduced by debonding of the metal decking and a reduction in the CMA contribution due to deformation of the steel frame.

- Providing the connections in the steel frame of the building remain intact, fire damaged composite slabs can still sustain a load greater than the required design ultimate load, although greatly increased deformations of the steel frame may render these slabs unserviceable.

6. References

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7. Acknowledgements

The authors gratefully acknowledge the support provided by the EPSRC, BRE and Comflor Ltd.