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The effect of combined thermal and force loads on the behaviour of reinforced concrete beams
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Fire resistance of composite deck slabs
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Fire resistance of composite deck slabs

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Synopsis

The fire resistance of composite deck slabs is presented in terms of their loadcarrying capacity, integrity, and thermal insulation. It is shown that 90 min fire resistance can be achieved with mesh reinforcement in the slab, provided the slab and reinforcement are continuous over a number of supports and imposed loads do not exceed 6.7 kN/m². This was demonstrated by large-scale tests, and a summary of available UK test data is made. In other cases of design, the fire engineering method may be used. This takes account of the reduced strength of the elements in fire.

Minimum slab depths are determined from thermal insulation requirements. The results of a series of unloaded tests on different slab depths and profile shapes are presented. This has led to a relaxation in minimum depths relative to previous guidance. The results of other fire tests, investigating (firstly) slab temperatures adjacent to beams and (secondly) thermal bowing of slabs, are also given in outline.

Introduction

It is estimated that the market for steel frames in commercial buildings in the UK is some 300 000 t p.a., and the majority of this involves steel decking as an integral part of the structural floor. The total decking usage for floors (as opposed to cladding) is close to 2 M m² annually.

'Composite construction' is the general term used in this context to denote the composite action of steel beams and concrete floors. 'Composite deck slabs' are those where profiled steel sheeting (or decking) acts as permanent formwork and as reinforcement to the concrete placed on top (Fig 1). Design of these composite deck slabs is now covered by BS 5950: *Part 4*[1]. When directly exposed to fire, the decking loses strength and the slab relies on mesh or additional reinforcement for its fire resistance.

Composite deck slabs are usually of 100 to 150mm overall depth, with spans ranging from 2.5 to 4.0m when the slab is not propped during the construction stage. Several decks are marketed (Fig 2) with heights between 46 and 76mm and sheet thicknesses between 0.8 and 1.5mm. The steel is galvanised for durability. A common feature of all these decks is the use of embossments to increase the mechanical bond between the concrete and steel.

The rapid increase in the use of this form of construction in the UK has occurred since 1980, although the first major project was the National Westminster Bank tower in London, completed in 1978. The largest current project is the Broadgate redevelopment in London, with total floor area of around 150 000m². However, the majority of 'composite' buildings are of floor area 5000 to 15 000 m².

Despite the fact that composite construction has been used in North America for 20 years, regulating authorities in the UK and in Europe were concerned about the lack of data on the fire resistance of composite slabs, which would satisfy European fire test requirements, now embodied in BS 476 *Part 20*[2] (formerly *Part 8*). In 1982, an important meeting was convened by representatives of the Greater London Council, involving deck manufacturers and designers, at which a plan of research and testing was first discussed.

In the 5 years since then a considerable amount of large-scale testing of composite deck slabs has been carried out by various research organisations in the UK. Further fire tests have been carried out in Europe[3]. This paper gives an opportunity to review these recent UK tests and to put forward guidance for designers.

The objective of the research in the UK was to show that, for the normal range of slab depths, spans, imposed loads and decking types, fire resistances of up to 90 min could be achieved with only mesh reinforcement in the concrete. This period of fire resistance would normally be required for most large commercial buildings (except in basement or storage areas). For greater

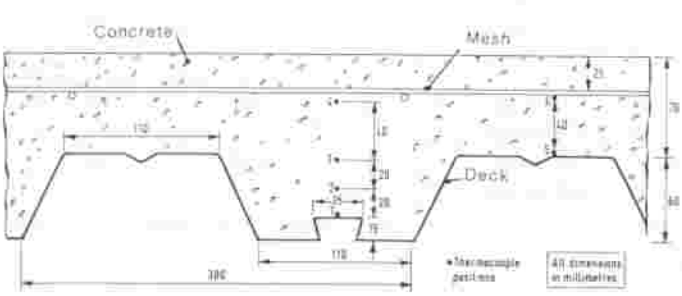

Fig 1. Section through a typical composite deck slab

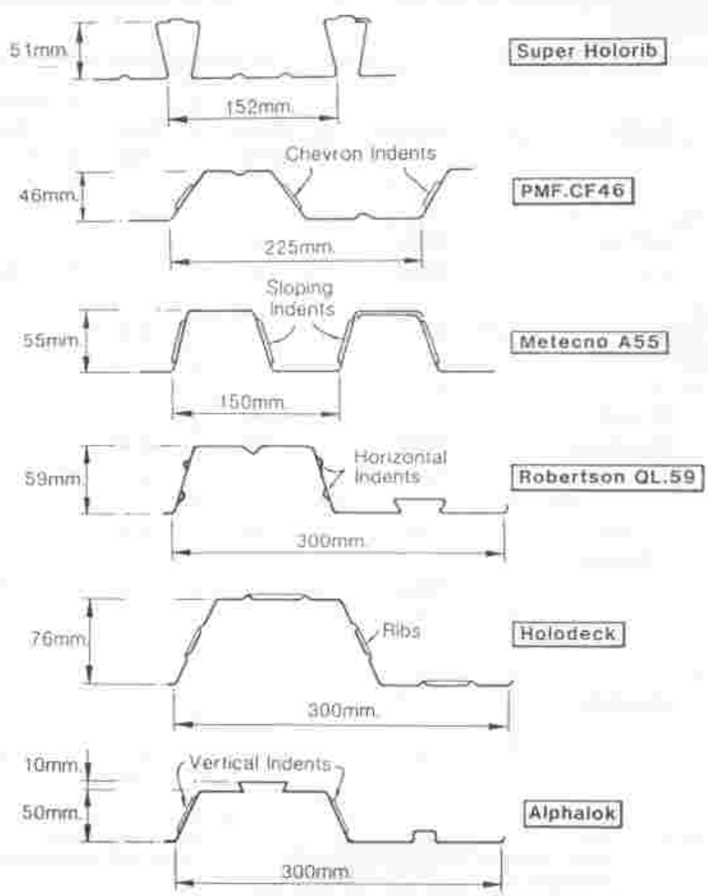

Fig 2. Profiles used in composite deck slabs subject to fire tests in the UK

periods of fire resistance, or spans or loads, additional reinforcement would be required, usually in the form of bars placed in the troughs.

The total cost of this research represents a small fraction of the potential sales of composite decking. Without this research, there would have been a marked loss in economy of this form of construction, as additional amounts of reinforcement or soffit fire protection would have been required for most new buildings.

Review of fire tests (before 1982)

Three criteria are imposed by the standard time-temperature fire resistance test specified in BS 476: *Part 20*[2]. These are stability (or strength under load), integrity to prevent passage of smoke or flame, and a limit on heat transmission to prevent excessive temperature rise on the upper surface of the slabs. The stability criterion is now covered by a deflection limit of span/20 or a maximum rate of deflection. Prior to the introduction of *Part 20*, a deflection limit of span/30 was used, although this limit has been relaxed, at least informally, for all tests on composite deck slabs since 1983.

Fire tests on floors are relatively expensive if realistic boundary conditions are examined, and are time-consuming because of the drying-out time of the concrete. Before 1982, few fire tests had been performed on composite deck slabs continuous over internal supports. Simply supported composite deck slabs with mesh reinforcement rarely demonstrated a fire resistance exceeding 30 to 60 min and, therefore, most of the existing tests had included additional reinforcing bars. These tests are summarised in Table 1.

The beneficial effect of continuity of composite deck slabs (provided by their mesh reinforcement) over internal supports was demonstrated by a fire test on the Hibond profile (by Metecno) which survived 78 min without reaching the limiting deflection of span/30. This used lightweight concrete (LWC) with an additional screed. Because of the constraints of the test furnace at TNO in the Netherlands, the slab comprised adjacent long and short spans of 3.0m and 1.5m, respectively. A jack force was applied at the end of the short span, in order to provide continuity.

The Underwriters' Laboratory (UL) in the USA has made extensive classifications[4] of the fire resistances of composite deck slabs. However, the fire test configuration used in North America permits development of internal restraint forces within the slab when heated by use of a boundary strong-frame. This is representative of the membrane action when one section of a slab is heated and constrained by its unheated neighbours. This action would occur in internal bays, but would be largely absent in edge bays of floors.

In European testing the beneficial effect of inplane restraint is neglected, and for this reason many of the UL data do not satisfy UK requirements. However, the fire resistance period should also be considered in the context of the Building Regulations. For buildings of similar size and use, a higher fire rating would generally be specified in the USA than in the UK. This tends to offset some of the more beneficial aspects of fire testing in North America.

The fire test data existing in 1982 were, therefore, considered to be inadequate for the economic design of composite deck slabs in the UK. It was apparent that there was a need for large-scale testing of composite floors incorporating appropriate boundary conditions. Any tests should be designed to be accommodated within the available furnace sizes of 6.6m × 4.3m at the Loss Prevention Council (then FIRTO), Borehamwood, and 4.3m × 3.0m at the Warrington Fire Research Centre.

Review of fire tests by CIRIA

Test series

The objectives of the test programme commissioned by the principal UK deck manufacturers, carried out by the Construction Industry Research & Information Association (CIRIA) between 1983 and 1985, were to:

(1) show that continuous composite deck slabs with standard mesh reinforcement could achieve a fire resistance of 90 min for spans up to 3.6 m and imposed loads up to 6.7 kN/m^2 (typical of most commercial buildings);

(2) confirm that slab deformations near failure would not adversely affect the performance of the supporting frame, so that the fire behaviour of the slab and frame could be considered separately;

(3) demonstrate that the long span/short span test could be used to represent the behaviour of a true continuous slab, this having implications for the cost of subsequent fire tests.

To avoid duplication of tests for each of the profile types shown in Fig 2, prior agreement of the regulating authorities (the GLC, in this case) to

TABLE 1—Summary of UK and European fire tests on composite deck slabs (pre-1982)

Profile	Concrete type[1]	Slab depth (mm)	Span[2] (m)	Imposed load (kN/m^2)	Reinforcement[3]	Surface temp. (C) After 1h	Surface temp. (C) After 1½h	Test period (min)
Holorib (UK)	NWC	150	4·2s	4·5	A98 mesh	45	—	60
	NWC	155	4·2s	10·0	A98 mesh Y20/trough	47	76	120
	NWC	130	4·2s	7·5	A98 mesh Y20/trough	48	84	90
Holorib (European)	NWC	150	4·5s	3·6	Y12/trough	—	—	60
	NWC	105	2·7s	4·8	mesh	—	—	55
	NWC	140	2·7s	10·0	mesh	—	—	60
Hibond-Metecno	LWC	120	3·0c	3·9 +1·3	Y10 @ 150 mm as mesh	38	—	78
	NWC	113	3·0s	2·5	A98 mesh	—	—	35
	NWC	140	2·4s	4·0	A142 mesh Y12/trough	—	—	90
Holodeck	NWC	150	4·0s	6·9	A98 mesh Y20/trough	92	105	90
	NWC	150	4·2s	3·3	A98 mesh Y12/trough	93	104	90

Notes: 1. NWC and LWC means normal and lightweight concrete, respectively.
2. simple span(s) or continuous span (c) in long span/short span configuration
3. includes 60 mm screed (non-structural)
A98 or 142 indicates 98 or 142 mm^2 mesh area/ m.
Y10, 12,20 indicate the diameters of high yield reinforcing bars, using 1 bar/trough.

TABLE 2—Summary of UK fire tests on composite deck slabs (post-1982)

Profile	Concrete type	Slab depth (mm)	Span (m)	Imposed load (kN/m^2)	Reinforcement	Surface temp. (C)		Test period (min)	Test ref.
						After 1h	After 1½h		
Robertson QL59	LWC	130	3·0s	6·7	A142 mesh	73	—	60	CIRIA 1
Robertson QL59	LWC	130	3·0c*	6·7	A142 mesh	70	100	105	CIRIA 2
Robertson QL59	LWC	130	3·0c	6·7	A142 mesh	95	110	90	CIRIA 3
Holorib (UK)	LWC	120	3·0c*	6·7	A142 mesh	60	100	90	CIRIA 4
PMF CF46	LWC	110	3·0c	5·25	Y5@225 as mesh	110	135	101	FRS-BSC1
Holorib (UK)	LWC	100	3·0c	5·75	Y5@150 as mesh	90	120	87	FRS-BSC2
PMF CF46	NWC	135	3·0c	6·75	Y5@225 as mesh	85	95	120 (136)	FRS-BSC3
Robertson QL59	NWC	140	3·6c*	6·7	A193 mesh	66	98	90	CIRIA 5
Metecno A55	NWC	140	3·6c*	6·7	A193 mesh	65	95	90	CIRIA 6

The tests are in chronological sequence from July 1983 until January 1986
+ failed prematurely because of loss of protection to beams
* test on long span/short span configuration

the proposed test series was obtained. One large-scale test would be carried out on a section of a floor comprising two equal spans to investigate the frame and slab interaction.

This would be supplemented by a series of five smaller-scale tests on the long span-short span slab configuration, as in Fig 3(a). These tests are summarised in Table 2 (CIRIA tests 2 to 6). All the tests would be performed on slab spans of 3m or 3.6m, between support beams, the main difference being the heavier mesh size used in the longer span tests.

To reduce the number of test variables the effect of the support conditions was investigated by keeping the same slab section and profile shape. For this purpose a typical trapezoidal profile was used (Robertson QL59). It was considered that all slabs with trapezoidal profiles designed for composite action would behave in a similar manner and that those with dovetail profiles would have a slightly higher fire resistance because of heat shielding of the ribs. This was confirmed later by identical tests using different deck profiles.

In CIRIA test 3 the beam layout and sizes were selected to be representative of current practice. A two-bay slab configuration was used consisting of two equal, 3 m spans. The grillage of support beams was itself supported on columns at 6m spacing. In an attempt to model the behaviour of a typical bay of a building of 8m width, which was twice as wide as the furnace, a sliding detail at the ends of the transverse beams was devised. This would not prevent pulling-in of these beams as the slab deflected.

The structural data for this test are summarised in CIRIA Report 107[5]. The 130mm-deep slab was in lightweight concrete (LWC) with A142 mesh at 25mm from the surface, as in Fig 1. The frame was provided with 25mm sprayed fire protection. For consistency, all the tests used an imposed load of 6.7kN/m^2 representing typical office and partition loading. In the above test 500kg steel weights were used, as shown in Fig 4. In the other tests, jacks were used both as applied loading and as the reaction at the end of the cantilever span to maintain continuity.

Testing of all the slabs was carried out after storing for 5 to 6 months

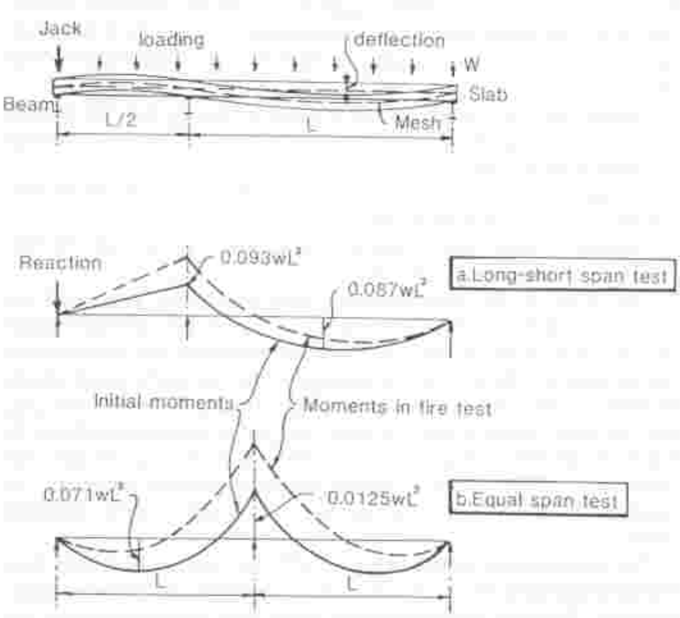

Fig 3. Test configuration and moments in fire tests on composite deck slabs

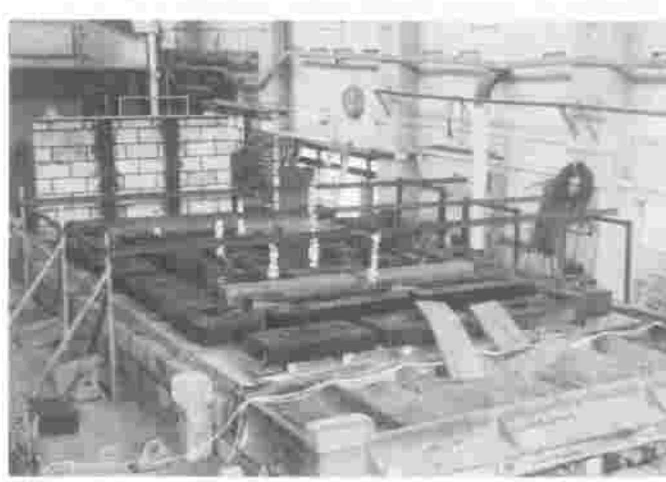

Fig 4. Loading on composite deck in a fire test

in dry conditions. The final moisture content of the lightweight (Lytag) concrete was 5.0 to 6.9 % by weight and that of the normal weight concrete was 3.5 to 4.5 %. The difference between the results is explained by the absorption of water by the Lytag aggregate. Expressed as a percentage by volume the results would be closer at 9 to 12 %. These figures are consistent with those given by Copier[6] who showed that, after 6 months in normal dry conditions, the moisture content would be around 20 % higher than under equilibrium conditions (after 2-3 years). Provided that the effect of excess moisture is considered in the interpretation of the temperatures recorded during the fire test, these moisture contents are not considered excessive. The concrete was in all cases of nominal grade 30 (cube strength 30 N/mm^2).

Test results

All the fire tests demonstrated that 90 min fire resistance could be achieved, provided that structural continuity of the composite deck slab could be developed. In test 3 no adverse effect on the behaviour of the support steelwork was observed. Indeed, there was a net outward movement of the slab resulting from thermal expansion which was greater than any tendency for the slab to pull in.

It was concluded that any 'cantenary' action of the frame and slab in resisting load was small and that slab behaviour was predominantly flexural.

The temperatures at various points in the slab section of Fig 1 are shown in Fig 5. The absence of any significant delay in the rise of temperature in the slab at around 100°C suggested that the moisture content was not excessive. At this temperature the heat applied to the slab would convert any moisture into steam rather than increasing the slab temperature. Mean surface temperatures were within the limits (140°C rise plus ambient temperature) of BS 476[2].

The 'heat-sink' effect of the concrete meant that the deck temperatures were locally up to 200°C less than the furnace temperature, an important observation as regards the loadcarrying capacity of the slab at elevated temperatures. Reinforcement temperatures were relatively low indicating that the full strength of the mesh could be mobilised. The temperature of the lower flanges of the composite beams indicated that the fire protection was just adequate to maintain their loadcarrying capacity at the end of the test period.

An important observation was the absence of severe debonding of the deck from the slab during the tests. A typical view inside the test furnace is shown in Fig 6. On cooling after the test, debonding did occur because of irreversible extension of the steel relative to the concrete. Any debonding during the test might develop an insulating layer beneath the concrete, thereby reducing the slab temperatures, but conversely, it would mean that the deck would have no contribution to flexural action. The influence of the deck on the loadcarrying capacity of the slab is considered in section 4.3.

For much of the early part of the tests the increase in deflection was a result of temperature-induced curvature, and only later due to structural weakening. It should be noted that absolute deflections, rather than deflections relative to the beam supports, were considered in assessing the limiting deflections of the slabs. This is a conservative interpretation of BS 476. At the end of test 3 (90 min), the rate of increase of deflection was 2mm/min, well below the limit in BS 476[2] which was calculated as 7.7mm/min. However, the absolute deflection was close to the span/20 limit. The slab recovered over 50 % of its deflection on cooling and survived the reload test.

The effect of changing the slab support conditions could be seen by

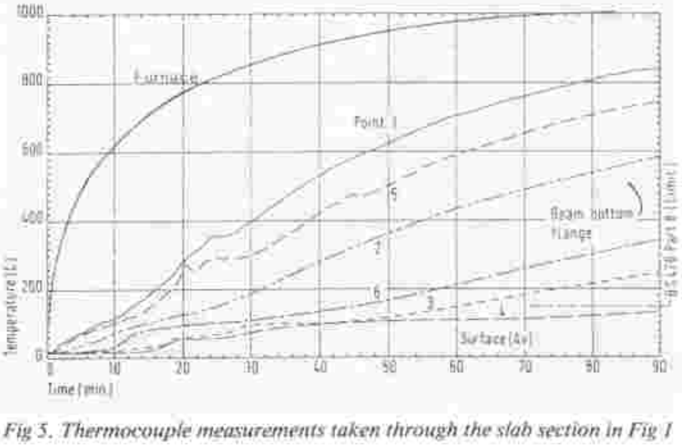

Fig 5. Thermocouple measurements taken through the slab section in Fig 1

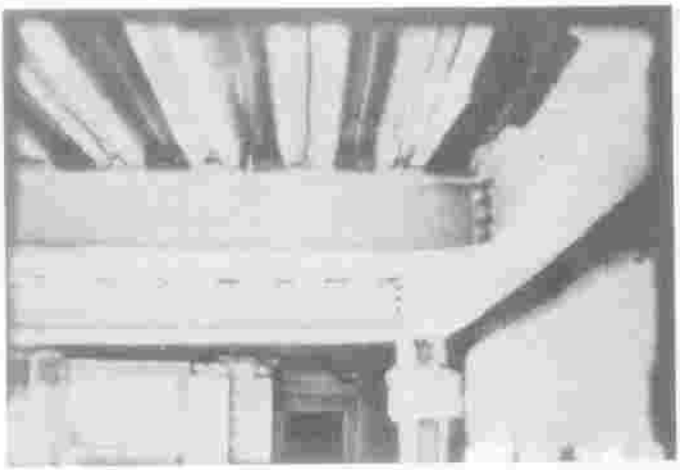

Fig 6. View inside the test furnace during a fire test

comparing this test (CIRIA test 3) with CIRIA tests 1 and 2. The simply supported slab (test 1) survived 60 min, and the long span/short span test (test 2), 105 min. Because the relatively small contribution of the support beams to the absolute deflection of this latter test, it was concluded that the slabs in tests 2 and 3 behaved in similar manner. Therefore, provided that continuity of the mesh reinforcement was developed, the statical system of the slab test was of less significance.

A fire test on a 120mm-deep Holorib slab (test 4) was carried out to confirm these results. A 90 min fire resistance was also achieved for this slab, despite being 10mm thinner than for the previous tests. This test demonstrated that slabs with the dovetail profile offer better insulation and shielding properties in a fire.

The test information was extended to spans of 3.6m by two tests (tests 5 and 6) on identical normal-weight concrete (NWC) slabs, differing only by the deck profile used. Both behaved in a similar manner with the tests terminated at 90 min. This confirmed the conclusion that slabs with trapezoidal deck profiles may be treated as a generic type in developing design recommendations for fire conditions.

The fire tests on these longer span slabs represented the first attempt at extending the recommendations for fire resistance to a more practical limit for the new generation of deeper profiles. The mesh reinforcement was increased to A193, but in principle the tests were similar to those previously performed. One novel feature was the use of shot-fired shear-connectors rather than welded studs. This did not adversely affect the fire performance of the test slabs.

Review of fire tests by FRS-BSC

Test series

A parallel test programme was carried out between 1984 and 1986 by the British Steel Corporation (BSC) supported by the Fire Research Station (FRS)) and the Department of the Environment. The results of this work have been collected in a report by British Steel Corporation[7]. The objectives were to:

(1) demonstrate the viability of the fire engineering method (see section 4.1) in determining fire behaviour of composite deck slabs, by carrying out large-scale fire tests;

(2) evaluate the effect of catenary forces (inplane restraint) on the fire resistance of composite deck slabs;

(3) develop efficient reinforcement arrangements to maximise the fire resistance of composite deck slabs.

The results of the three large-scale tests which were performed are summarised in Table 2 (FRS-BSC tests 1 to 3). All the tests were performed on continuous composite deck slabs intended to have 90 min fire resistance (by calculation). The test load was adjusted so that the reinforcement would be fully stressed at this fire resistance period. The test floor was 7.0m long and the beam supports were 3m apart. The test arrangement is shown in Fig 7. To develop catenary forces a system of tensioning the slab was included. A short cantilever section developed further continuity. In principle, the other features of the tests were similar to those described earlier.

The reinforcement pattern consisted of 5mm diameter high yield bars welded in the form of a mesh and cranked at around 0.7m from the beam supports. The mesh was positioned at 25mm from the slab surface at the supports and on the surface deck in midspan. The longitudinal bars in the mesh were located over the deepest section of concrete and were lapped over the beam supports, thereby giving the maximum fire resistance (see Fig 8).

In two of the tests for trapezoidal profile (PMF CF 46 (shown in Fig 2)) was used, and in the third, the profile Holorib was used. Both normal- (NWC) and lightweight concrete (LWC) were included in the test series. The beams were provided with board- type fire protection. Testing was carried out after storing for 6-7 months in dry conditions, and measured moisture contents of the concrete were 3.5 % for NWC and 5.5 % for LWC. The concrete was again a nominal grade 30.

One important observation of the test specimens was made. Because of the deflection of the deck arising from the weight of concrete placed on it during construction (as would be expected in practice), the slab depth in midspan was some 10 % greater than the desired value. This was a significant factor in interpreting the results of the fire tests.

Test results

In the fire tests on the slabs with trapezoidal deck profiles (FRS-BSC tests 1 and 3), fire resistance times in excess of 90 min were achieved. Test 3 reached the limiting deflection criterion of span/20 at 120 min but the test was continued until the accelerating rate of deflection indicated that failure was imminent. The test was terminated at 136 min (see Table 2). Both tests survived the reload requirement.

The surface temperature in test 1 approached the BS 476 limit at 90 min, whereas in test 3 it was apparent that the slab depth exceeded that required for minimum thermal insulation in a fire. This test on a normal weight concrete confirmed the observations made in CIRIA tests 5 and 6 that the previous requirements for minimum slab depth had been too conservative. Test 2 on the Holorib slab in LWC performed as expected from a thermal insulation point of view. Indeed, at 100mm, this was the minimum depth of the slabs tested.

In all the tests the deflection response was roughly linear with time (1 to 2mm/min) until close to the end of the tests when there was a more rapid increase in deflection. Test 2 was terminated after 87 min, well before failure of the slab, because of loss of fire protection to one of the beams. From the deflection response of the test slab it was concluded that a 90 min fire resistance would have been achieved.

The tensioning system was not used in any of the tests because, as was found in the CIRIA tests, the net movement of the edge beams was outwards. These tensile restraints were intended to act only as the slabs pulled in at large deflections, reflecting the possible 'catenary' action of real floors. However, the overall temperature expansion of the slab was the dominant effect up to the relatively small vertical deflection limit imposed by BS 476: *Part 20*.

The reinforcement pattern used in these tests was such that the moment capacity of the slabs was similar in both hogging and sagging. This represented a much more efficient distribution of reinforcement, despite being similar in area to the standard mesh used in the CIRIA tests, and contributed to an improvement in the fire resistance of these slabs.

An observation in tests 1 and 3 was that some of the high yield bars over the supports apparently reached their ultimate tensile strain and broke. The ultimate tensile strain of steel increases with temperature and might be expected to be of the order of 2 % at the temperatures and deformations experienced at the level of the reinforcement. Despite the resulting loss in moment capacity at the supports, this did not lead to premature failure of the slabs. Nevertheless this had highlighted the need to be cautious about

Fig 7. Arrangement of load in FRS-BSC tests

Fig 8. Cranked reinforcement used in FRS-BSC tests

the use of high tensile drawn wire reinforcement at periods of fire resistance exceeding 90 min. As in the previous fire tests, there was little evidence of debonding of the deck during the tests.

Fire engineering method

Analysis method

A method of calculating the fire resistance of concrete structures was first proposed in the IStructE report *Design and detailing of concrete structures for fire resistance*[8]. In principle, this method uses measurements of the temperature rise in various concrete sections, together with the known behaviour of steel and concrete at elevated temperatures, to calculate the reduced moment capacity of the section. The appropriate fire resistance period of simple beams or floors can be ascertained by comparing the reduced moment capacity of the element to the applied moment calculated with reduced safety factors. For continuous beams, the hogging and sagging capacities are added.

The SCI report[9] has modified this method to reflect the fire behaviour of composite deck slabs. As the temperature rise within the concrete is, to some extent, dependent on the shape of the deck profile, a modification factor was introduced to consider the heat flow to a given point from the adjacent exposed surfaces. However, reinforcing bars positioned within the troughs of the deck would be expected to reach their limiting strength after a longer period than the equivalent reinforced concrete ribbed slab, because of the absence of spalling. Mesh reinforcement positioned within the slab topping could be expected to be well-insulated from the fire. Conversely, the bottom 20mm of the concrete in the troughs would usually be neglected in fire engineering calculations, since the concrete strength would have reduced considerably.

In continuous slabs with standard mesh reinforcement, there is a redistribution of moment during a fire from the midspan area to the supports, as shown in Fig 3(b). This is both as a result of thermally induced curvature and weakening of the deck before the other elements. At failure, the moment capacities of the slab in hogging and sagging (negative and positive moment) may be combined, as in an equivalent plastic failure mechanism. The hogging capacity would remain close to its 'cold- state' value, whereas the sagging capacity would have diminished considerably.

Properties of materials

The behaviour of concrete and steel at elevated temperatures is well known[8]. Typically, normal weight concrete loses half its strength at around 700°C and steel at around 550°C. However, an important consideration is the strain at which the strengths of these materials are assessed. In most composite deck slabs, steel strains in excess of 2 % would be experienced at the limiting slab deflection of span/20. Compressive strains in the concrete would be about 0.5 %.

Conventionally, the strength data for steelwork at elevated temperatures is given at 0.5 % strain, a value partly influenced by the desire to prevent the fire protection to the members from becoming dislodged. However, for steel there is a steady increase in strength between 0.5 % and 2 % strain in the important temperature range of 400°C to 700°C[10]. At 550°C, the tensile strength at 2 % strain is 25 % higher than at 0.5 %. This is true for both structural steel and reinforcing bars, but there is not yet sufficient data on cold-formed steel (such as decking) to quantify this increase precisely. At temperatures above 800°C, a residual strength of 5 % of the

TABLE 3—Comparison of moment capacities (based on measured temperatures and properties) with the applied moments in fire tests

Test no. (see Table 2)	Section capacities (kNm/m)			Plastic capacity of slab (kNm/m)		Applied moment[4] M_o(kNm/m)
	Hogging (M_n)	Sagging (M_p)[1]		$M_u = M_p + 0{\cdot}45\ M_n$		
	Mesh[2]	Mesh	5% deck[3]	Mesh	Mesh & deck	
FRS-BSC1	8·3	4·2	2·2	7·9	10·1	7·9
FRS-BSC2	9·9	4·6	2·5	9·0	11·5	10·3
FRS-BSC3	10·4	4·6	2·4	9·3	11·7	12·3
CIRIA 3	8·0	3·4	2·0	7·0	9·0	9·1
CIRIA 4	6·6	2·9	2·7	5·9	8·6	9·3
CIRIA 5	11·1	4·5	2·4	9·6	12·2	14·3
CIRIA 6	11·6	4·5	3·3	9·8	13·1	14·5

1. Sagging moment capacity evaluated from the measured section depth in midspan.
2. 'Mesh' means the moment capacity of the mesh or reinforcement using the measured yield strength.
3. Additional moment capacity evaluated using 5% of the tensile capacity of the deck.
4. Applied moment on simple span. Span based on beam spacing minus 0·7 times beam width.

initial yield strength is appropriate for all grades of steel. High temperature data for reinforcing bars is given by Holmes *et al*[11].

Lightweight concrete has superior properties in fire in comparison to normal weight concrete. Firstly, its thermal conductivity is lower, meaning that internal temperatures would be lower and, secondly, it loses strength less rapidly at higher temperatures. The data are given in refs. 8 and 9. It is possible also that limestone aggregate concrete is slightly better than gravel aggregate concrete in these respects.

Analysis of fire behaviour of test slabs

Fire engineering analyses have been carried out for all the fire tests on continuous composite deck slabs, using the material properties given in refs. 9 and 10. In all these tests, the results of the fire engineering calculations indicated considerably lower strengths (or fire resistance periods) than the apparent strengths at the end of the tests. This could be explained by a number of factors:

(1) The decking could have played a role in increasing the positive (sagging) moment resistance of the slab.

(2) The strains in the reinforcing steel, particularly in the negative (hogging) moment region, were much greater than 0.5 %, giving a potential increase in strength over the data in refs. 9 and 10.

(3) Measured material strengths were greater than their characteristic values.

(4) Measured temperatures within the sections were generally lower than those assumed in the calculations.

(5) The slab depths in midspan exceeded their nominal depths (because of deflection of the deck under the self-weight of the concrete).

It should be noted that the mean strength of the steel mesh in the tests was 610 N/mm^2, some 25 % greater than the characteristic value of 485 N/mm^2. (BS 8110[12] now specifies 460/mm^2 for all high yield reinforcement.) Measured concrete strengths were 40 to 45 N/mm^2 (nominal grade 30).

The condition for plastic failure at an end span of a continuous composite deck slab is given by the formula:

$$M_p + 0.5\ M_n\ (1 - \frac{M_n}{8M_o}) \geq M_o$$

where M_p and M_n are the sagging (positive) and hogging (negative) moment capacities of the composite section, respectively, and M_o is the free bending moment applied to the simply supported slab in fire conditions. The second term approximates to a value of $0{\cdot}45M_n$.

The moment capacities of the sections have been redetermined using measured temperatures and material properties. These results are summarised in Table 3. The plastic moment capacity of the composite deck slabs has been evaluated using the above formula. In interpreting the test results, account was also taken of the actual depth of the slab in midspan (see note 5). In the FRS-BSC tests this depth was recorded, but in the other tests it has been assumed that the actual depths were 10 % greater than the nominal depths (i.e. at the supports).

Comparison of the theoretical failure capacities of the continuous slabs with the test moments indicates that there is still a shortfall in capacity for most of the tests (see Table 3). Better agreement is obtained when a nominal allowance of 5 % of the tensile capacity of the deck (at room temperature) is included in determining the sagging moment capacity of the slab. Clearly, this assumes that there is sufficient contact between the deck and the concrete at the later stages of a fire to develop this tensile force. However, the required bond is very small.

The conclusion from these fire engineering calculations is that there are many beneficial factors that are not taken into account and that make the method relatively conservative with respect to the test results. The test fire resistances can, in general, be justified only by including a contribution from the tensile strength of the decking in a fire engineering calculation. A residual strength of 5 % would appear to be reasonable given the performance of steel at high temperatures and the uneven temperature distribution around the deck profile.

Review of fire tests to determine minimum slab depths

Two fire tests were carried out by the Fire Research Station in 1986 to determine the thermal characteristics of 40 different configurations of composite deck slabs. The individual modules tested were each nominally 1m x 0.65m and were exposed to BS 476: *Part 20* heating. In each test there were two comprehensively thermocoupled assemblies of 10 modules each, which were cast together to form an integral unit (Fig 9). The tests were carried out in the large LPC furnace.

The variables used in the parametric study included overall slab depth, the type of concrete, and the lateral position of the reinforcing mesh. The slab depths ranged from 90 to 196mm to represent the minimum depths recommended in the SCI report[9] for insulation corresponding to fire resistances of ½ to 4 h. The deck types were chosen to be representative of current practice. A full range of tests in both normal (gravel aggregate) and lightweight (Lytag aggregate) concrete was carried out for the Holorib and PMF CF46 profiles. For comparison a number of slabs with the

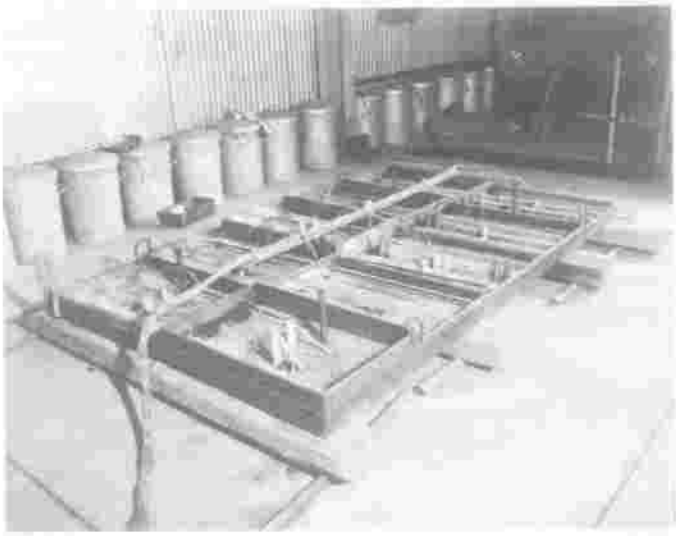

Fig 9. Arrangement of specimens in the FRS thermal insulation tests

Metecno-Hibond profile were also tested. The test series is summarised in Table 4.

The temperatures recorded at various points within a typical test slab are shown in Fig 10 (a) and (b). Of most interest are the temperatures of the upper surface of the slabs, since this often dictates the slab depth that can be used in design, and the temperatures of the mesh reinforcement. In all tests the mesh was placed directly on the deck.

TABLE 4—Summary of FRS fire insulation tests

Profile	Concrete	Slab depths (mm) tested
PMF CF46	LWC	101, 111, 121, 131, 170
PMF CF46	NWC	111, 136, 151, 161, 196
Holorib	LWC	90, 105, 115, 150
Holorib	NWC	90, 110, 125, 170
Metecno-Hibond	LWC	110, 120, 140, 185

In determining the temperatures on, and close to, the upper surface of the slab, account should be taken of the 'dwell' in temperature at around 100°C resulting from the vaporisation of free moisture. It is conservative to reduce the temperature-time curves by the total apparent 'dwell time', although a certain percentage of moisture is present in all concrete. The dwell time would normally be between 15 and 30 min for surface temperatures (where any free moisture would collect) and 5-20 min internally.

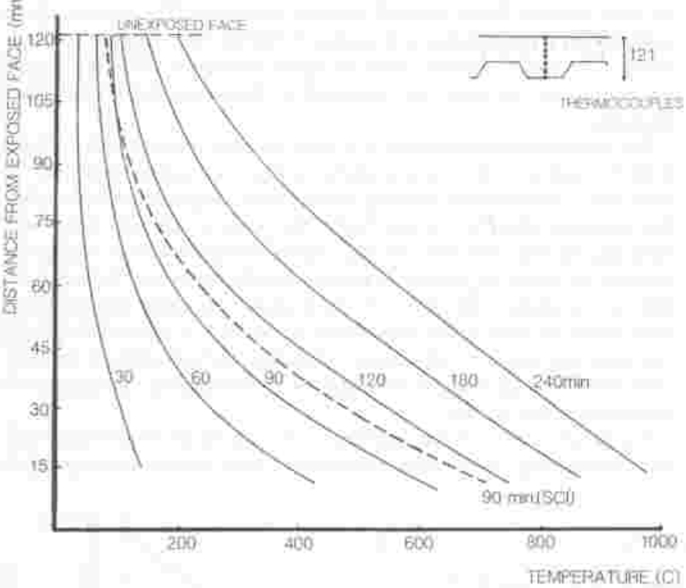

Fig 10(a). Temperature distribution above trough of composite deck slab incorporating LWC and CF46 profile

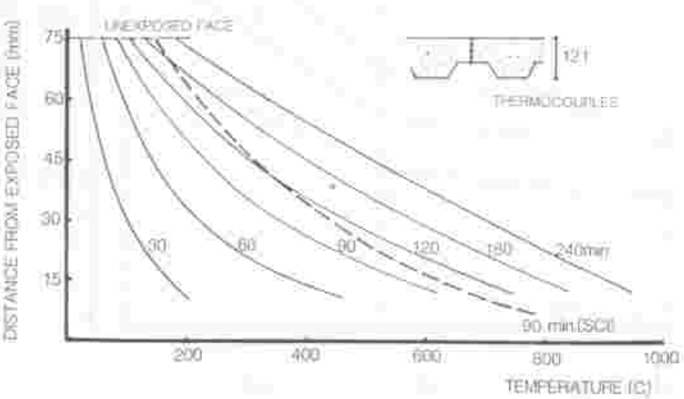

Fig 10(b). Temperature distribution above crest of composite deck slab incorporating LWC and CF46 profile

The time to reach the unexposed surface temperature limit in BS 476: *Part 20*, after reduction of the dwell time, is summarised for all the tests in Fig 11. In most cases the appropriate temperature limit is 140°C average temperature rise above ambient. Traditionally, results have been expressed in terms of the depth of concrete over the deck, in the case of trapezoidal profiles, or the total depth in the case of dovetail profiles.

An alternative form of presentation of the test results is in terms of the mean depth of concrete, which is the approach used by the ECCS[7]. The effect of this is to eliminate the influence of the profile shape when interpreting the test results, as can be seen in Fig 11.

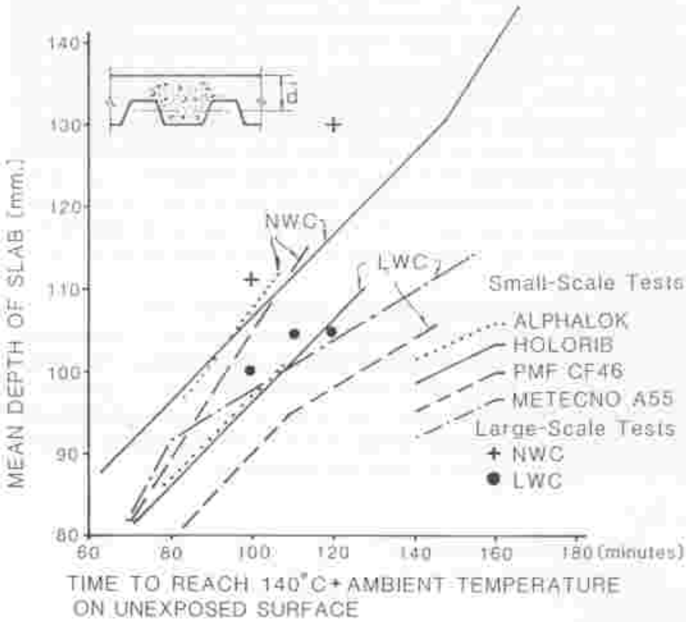

Fig 11. FRS insulation tests expressed in terms of mean slab depth

The curves for lightweight concrete (LWC) are displaced by around 30 min relative to those for normal weight concrete for the same mean slab depth, demonstrating the greater insulating properties of LWC. Also presented in Fig 11 are the results obtained from the large-scale tests reviewed earlier and for thermal profile tests recently carried out by SCI on the Alphalok profile.

These results now give greater confidence in the selection of appropriate data for design. To be consistent with existing UK practice, the criteria for minimum slab depths (i.e. concrete depth over trapezoidal profiles, or total concrete depth for dovetail profiles) are retained, but the data modified.

These minimum slab depths are presented in Table 5. This represents a considerable relaxation over ref.9, particularly for normal weight concrete. The SCI publication will be updated shortly to take account of this work.

TABLE 5—Different means of expressing minimum slab depths (mm) to satisfy the insulation criterion in a fire resistance test

Profile type	Criterion for minimum dimension	Concrete type	BS476 insulation limit (h)					
			½	1	1½	2	3	4
Trapezoidal	Depth above top of profile	LWC	(50)	60	70	80	100	115
		NWC	60	70	80	95	115	130
Dovetail	Total slab depth	LWC	(90)	(90)	100	115	130	150
		NWC	(90)	(90)	110	125	145	165
Any	Mean slab depth (Proposal)	LWC	(65)	80	95	110	130	145
		NWC	75	90	105	125	150	170
Any	Mean slab (ECCS(3))	NWC	(60)	70	80	100	130	150

( ) Minimum depth over profile is 50 mm

For comparison, the appropriate mean slab depths are also given in Table 5. It should be noted that the mean slab depths in the ECCS Technical Note[3] are significantly lower than those given by this research and are not recommended for design use.

Thermal profiles through the slab are used in the fire engineering methods. Representative sections through the concrete are at the centreline of the trough and at the centreline of the crest of the profile (as in Fig 10). The thermal profiles determined from the small-and large-scale fire tests have been compared to the existing design curves given in refs. 8 and 9.

One observation from the tests was that the temperatures through the section at the centreline of the trough were not significantly greater than through a solid section of the same depth, provided that the average trough width exceeded the slab depth. This is the case for most of the modern profiles where trough widths are 120 to 150mm. Conversely, temperatures directly over the crest of the profile were much lower than indicated by the minimum depth of concrete considered as a solid slab of this depth.

The results of some of these tests have been summarised in the form of Fig 12. The data have been corrected to eliminate the effect of moisture. In the future, it should be possible to prepare thermal profiles for all slab depths, concrete types, and profile shapes. However, in the interim, it is proposed to use the design data for solid slabs (Table 4.5 in ref. 9).

Analysis of thermal profiles through composite deck slabs

Various computer programs based on non-steady heat flow in solid bodies have been developed. By introducing the appropriate thermal conductivity, specific heat and density values for the various materials, it is possible to model a particular concrete steel cross-section in a series of finite elements. The influence of the standard temperature-time curve[2] applied to the soffit of a floor can be studied by time-step integration.

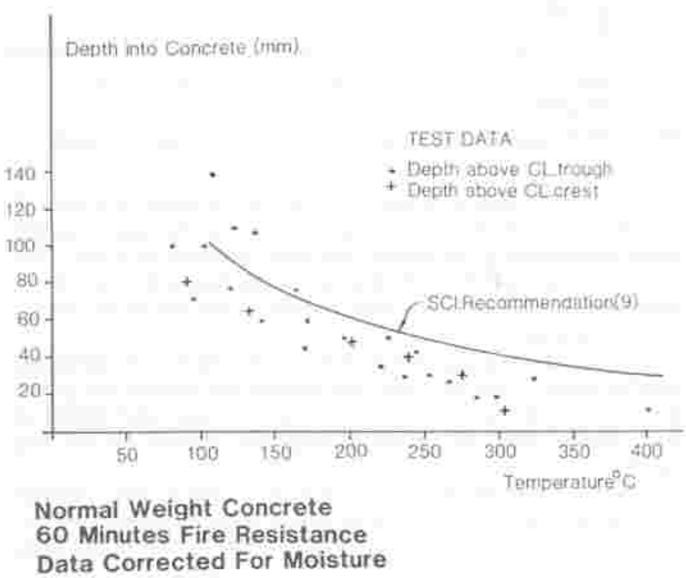

Fig 12. Measured temperatures in concrete compared to recommendation[9]

A computer simulation of heat flow through a composite deck slab has been attempted using the program FIRES-T[13]. This was carried out at the University of Aston. For comparison with the test results, an analysis was made of a 140mm-deep normal weight concrete slab (CIRIA test 5 in Table 2), designed for 90 min fire resistance. The resulting temperature profile through the cross-section is shown in Fig 13, and these are compared with measurements made during the test.

Of interest are the relatively low temperatures at certain points on the deck profile (particularly the upper curves of the profile), and the 'flow' of temperature reducing the effect of the thinnest section of concrete. Comparison between the model and measurements was good. Surface temperatures were generally higher than those measured but still within the BS476 limits. It should be noted that heat flow models do not take into account the effect of moisture which means that they will tend to overpredict temperatures in concrete.

Additional tests

Tests on slabs supported by steel beams

Although most of the fire tests reported earlier incorporated beams to support the composite deck slabs, there has been one recent test in which

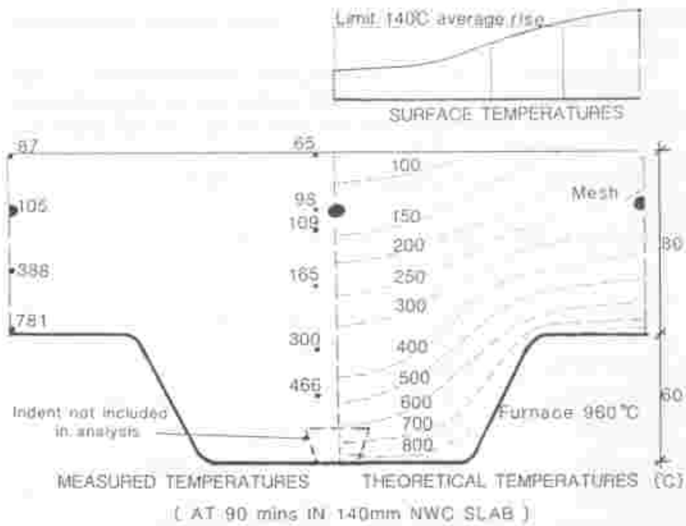

Fig 13. Comparison of measured and theoretical temperatures in a composite deck slab

a special study was made of the insulating effect of the beam on the temperatures within the adjacent slab.

A fire test designed by the Steel Construction Institute and funded by BSC and FRS was carried out in 1987 on two unloaded beams supporting composite deck slabs. Lightweight and normal weight concrete were used, together with open trapezoidal and dovetail decks. The beams were provided with sprayed fire protection. Temperatures were measured on the fire protected beams and at various positions in the slabs. An intresting feature of the protection was that, in two positions on each beam, the void between the deck and top flange was left without protection. This is contrary to common site practice where it is normal to fill the voids beneath the deck profile with either mineral wool or spray protection. The top flange temperature of the beam was measured at filled void and unfilled void positions.

The variation of temperature at a given point in the slab with distance from the centre of the beams is shown in Fig 14. It can be seen that the beam has a considerable shielding effect on the temperatures in the slab. The shielding takes two forms: firstly, the beam simply acts as a shield reducing the heat flow into the slab, and secondly, moisture within the concrete is driven by vapour pressure towards the beam. Temperature plateaux at about 100C were observed above the beam and were not observed to the same extent away from the beam. It is concluded that the relatively cold region of slab adjacent to the beam ensures that the concrete compression flange may be fully utilised in establishing the strength of composite beams in fire.

The measurements of temperature on the top flange of the beam were very illuminating. The unfilled voids caused little increase in temperature

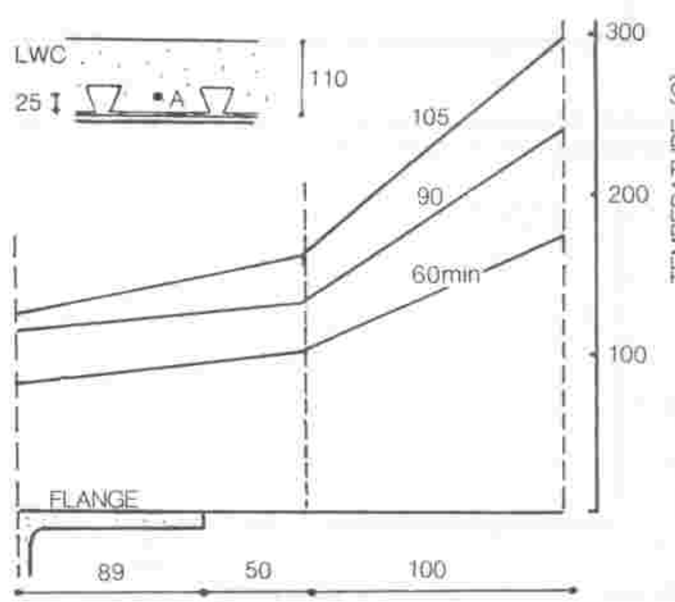

Fig 14. Temperatures at point A at positions adjacent to steel beam

for the dovetail decks and about 16°C increase for the open trapezoidal deck. These temperatures would have little effect on the moment capacity, as it is the lower flange temperature which determines the reduced strength of the composite beam. It is concluded that the practice of filling voids between steel decks and the supporting composite beams is probably unnecessary.

Thermal bowing
Furnace tests on floors usually adopt the standard fire exposure[2] which represents a 'cellulose' fire. With more plastics being introduced into the contents and fabric of some buildings, the fire exposure may be more severe, especially in the early stage of a fully developed fire, and the temperatures may correspond more closely to a 'hydrocarbon' fire. Composite deck slabs have not been tested subject to the hydrocarbon fire exposure, but an indication of their performance could be obtained from fire tests on solid concrete slabs if it were assumed that the decking has a negligible effect in the later stages of a test. The observation has already been made from the loaded fire tests that thermal bowing can be a significant part of the total deflection of a floor. Thermal bowing is a measure of the temperature-induced strains through the concrete or composite section.

FRS has undertaken a programme of furnace tests[14] on pairs of independently mounted, simply supported solid concrete floor strips (each 1m wide, spanning 4.5m and exposed over a 4m length) to investigate the magnitude of midspan deflection when the two different fire exposures are used. The NPV time-temperature curve[15] was used to simulate the hydrocarbon fire exposure. Some results for unloaded slabs of 150mm and 250mm depth are shown in Fig 15. This demonstrates the increased deflections resulting from the more severe NPV exposure, in comparison to the standard fire. The superior behaviour of lightweight aggregate concrete (Lytag/PFA) is also illustrated. The reduced deflections are due mainly to the lower coefficient of thermal expansion and lower thermal conductivity of lightweight concrete.

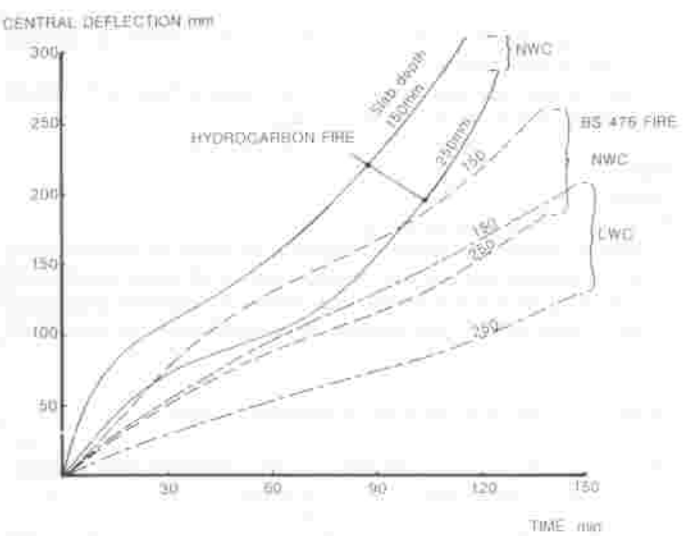

Fig 15. Effect of heating regime and concrete type on thermal bowing of concrte slabs

Comment on repairability of composite deck slabs after a fire
Fire is a rare occurrence, and its effects are rarely as serious as experienced in a fire test. Repairability is not a criterion of the fire resistance test, which is concerned with strength and stability in a fire. This applies to all materials, and some repairs should generally be envisaged after a moderate fire. In reinforced concrete it would normally be necessary to replace the cover to the reinforcement which may have partially lost its strength or spalled. In composite deck slabs it would normally be necessary to cut-away any sections of decking which have lost contact or debonded from the soffit of the slab. Some permanent deflection of all concrete or composite deck slabs would be experienced after a severe fire. Fig 16 shows a typical slab after a 90 min fire resistance test.

Although observations made during fire tests on composite deck slabs indicate that any debonding tends only to be localised, further debonding is often apparent after a fire test. This is because of irreversible extension of the deck in relation to the concrete which causes the deck to pull away on cooling. Therefore, after a moderate/severe fire, it would be necessary to remove the damaged deck and replace it, e.g. by reinforcing bars attached by shot fired clips between the concrete ribs. The bars could then be gunited *in situ* to achieve appropriate bond. In severely damaged or deformed slabs, replacement of the complete section of slab between the steel beams should be considered. Some guidance on the reinstatement of fire- damaged structures is given in refs. 16 and 17.

Fig 16. Composite deck slab after a 90 min fire resistance test

Design recommendations
The fire resistant design of composite deck slabs has been considered on the basis of large-scale loaded tests, to determine their limiting strength, and small- scale unloaded tests to establish the appropriate minimum slab depths[2]. The strength of composite deck slabs is largely a function of their span-to-depth ratio and the amount of 'fire-reinforcement.' If standard mesh reinforcement is used, the slab depth may be controlled by either strength or insulation requirements.

In general, the large-scale tests have shown that composite deck slabs with trapezoidal profiles, designed to satisfy the appropriate thermal insulation limit, are adequately strong in fire, provided mesh reinforcement

TABLE 6—Simplified rules for fire resistance of composite deck slabs[14]

Max. span (m)	Fire rating (h)	Minimum dimensions			Mesh size
		Sheet thickness (mm)	Slab depth (mm)		
			NWC	LWC	
2·7	1	0·8	130	120	A142
3·0	1	0·9	130	120	A142
	1½	0·9	140	130	A142
3·6	1	1·0	130	120	A193
	1½	1·0	140	130	A193

(A) Trapezoidal profiles (depth not exceeding 60 mm)

Max. span (m)	Fire rating (h)	Minimum dimensions			Mesh size
		Sheet thickness (mm)	Slab depth (mm)		
			NWC	LWC	
2·5	1	0·8	100	100	A142
	1½	0·8	110	105	A142
3·0	1	0·9	120	110	A142
	1½	0·9	130	120	A142
3·6	1	1·0	125	120	A193
	1½	1·0	135	125	A193

(B) Dovetail profiles (depth not exceeding 50 mm)

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(5) No significant reduction in ultimate force load capacity of indeterminate beams occurred as a result of coexisting thermal loading.

Acknowledgement

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of a minimum size is detailed. For slabs with dovetail profiles, minimum slab depths are controlled more by the strength than the insulation limit. Therefore, by increasing the amount of reinforcement, the slab depth may be reduced, if so desired.

The principal conclusion of the research is the adequacy of continuous composite deck slabs for 90 min fire resistance for spans up to 3.6m and imposed loads up to 6.7kN/m^2. This reduces to 30 min if adequate continuity cannot be provided at one or more supports. The mesh reinforcement is A142 or A193, depending on the span of the slab. These design recommendations are summarised in Table 6, from ref. 18, and are consistent with the test results in Table 2.

When periods of fire resistance greater than 90 min are needed, the fire engineering method should be used[9], consistent with the minimum slab depths in Table 5. The FRS-BSC tests have shown that optimum detailing of reinforcement may be used in this method. Any relaxation in the requirement proposed in Tables 5 and 6 (and ref. 18) should be justified by further fire tests or fire engineering calculations.

Acknowledgements

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